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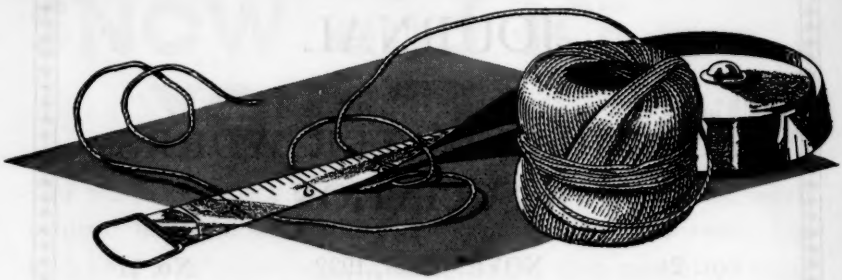

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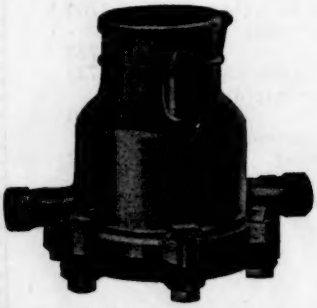

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WATER DIVERSION BETWEEN DRAINAGE BASINS

By Robert E. Horton[1]

The object of this paper is to discuss problems—economic, engineering and legal—which arise when diversion of a stream is undertaken. The term "diversion" is here used in the broad sense of permanent abstraction of water from a given stream so that it does not return to the stream within the limits of the divertor's property. The water diverted may be taken across a watershed line into another drainage basin, or it may be transported to the seacoast and discharged into the ocean, or to a certain extent it may be used consumptively in industries or for irrigation purposes and returned to the air as vapor.

If the stream from which diversion takes place neither becomes nor crosses a state boundary below the point of diversion, complete jurisdiction over legal aspects of the diversion may be exercised by a single sovereignty or state, except in so far as navigation is affected, in which cases federal jurisdiction prevails.

If the stream enters another state below the point of diversion or if its thread forms the boundary between two states, then diversion in one of these or in another state upstream does not come within the jurisdiction of any single state or sovereignty and legal problems arise of a somewhat different nature from those in the first instance described.

[1] Consulting Hydraulic Engineer, Albany, N. Y.

The divertor state has no control or jurisdiction over the stream beyond its own state boundaries and the rights of downstream states or their inhabitants in such cases can be determined only by the United States Supreme Court. Aside from this legal difference, problems of diversion between drainage basins are of much the same nature whether state boundaries are crossed or not. This discussion will be chiefly directed to major diversions from one drainage basin to another. Much of the discussion is equally applicable to local diversions where the water is returned to the same stream at some point below the point of diversion, but outside or beyond the limits of the divertor's property.

Attention will be confined mainly to conditions in the eastern United States, although diversion for irrigation is not altogether in a class by itself in spite of the fact that a different system of water law often prevails in irrigation regions. A larger portion of the water is used consumptively in connection with irrigation diversions than in case of diversions for most other purposes. In spite of the fact that in irrigation diversions part of the water is returned to the air through evaporative processes, mainly transpiration by plants, economic and engineering phases of irrigation diversions are in many respects similar to those of other diversions.

Much has appeared recently in engineering literature on the subject of the legal aspects of diversion, particularly with reference to water supply uses, while the engineering and economic aspects of diversion of streams have received less attention than they deserve.

Diversion of water from one drainage basin to another may be economically desirable in connection with almost any one of the many beneficial uses which a river may serve. In the present paper some mention will be made of the various uses of streams, the various purposes for which diversion may be desirable, the economic relations of diversion to other uses of the stream, the hydraulic problems involved by diversion, and the necessity of a comprehensive plan of stream utilization, in any event, before diversion is undertaken.

This leads of necessity to a consideration of what in water works parlance is described as the "allocation" of waters, although the term is here used in a much broader sense than that ordinarily prevailing. It also involves a discussion of stream flow regulation in its relation not only to diversion but to the fullest economic utilization of the stream for other purposes as well.

Some attention is given to legal questions, although the author

has no intention of setting up an imposing array of case citations. That may be left to legal pundits or others with a flair for the "niceties of the law."

Finally, particular attention is drected to different methods of diversion, the underlying idea being that in each particular case that method should be utilized which fits in best with a general plan or program for the conservation and utilization of the entire water resources of the stream, with due regard to all the economic uses it may serve.

PRINCIPLES OF RIVER CONSERVANCY

Conservation in case of rivers has come to mean not only prevention of waste but economic utilization. Unlike a mine which, once exhausted, cannot be restored, a river, like Tennyson's brook, flows on forever. Its youth is renewed like the eagle's as it passes through the successive phases of the hydrologic cycle year by year. To permit a river to "flow unvexed to the sea" without beneficial use is waste.

A river or stream may serve, and most larger streams of the United States do serve, a variety of uses. These may include:

1. Fisheries and related industries.
2. Transportation.
3. Irrigation and agricultural uses.
4. Sanitary uses, including especially the removal and elimination (by self-purification) of organic and industrial wastes.
5. Public and industrial water supply.
6. Recreational uses.
7. Water power production.
8. Regulation and flood prevention.

These uses have been mentioned somewhat in the historical order of their development, without regard to their relative importance, which may, indeed, vary through a wide extent in case of different streams. Too often it appears that one particular use of a stream has been singled out as of paramount importance or has been developed without regard to other potential uses and without any plan looking to the best utilization of the stream for all available purposes. This sometimes results in partial, incomplete and misfit development of the stream and the growth of conditions which have hindered or inhibited the best economic use of the stream.

The author does not believe that any one use is always paramount or should dominate to the exclusion of all other uses. All the uses

listed, and perhaps additional uses, should be considered in devising a plan or program for the conservation and development of a stream. To use an illustration, something akin to city planning is needed for every important stream or river.

The author cannot improve on the clear statement of the needs and requirements of conservation of water resources contained in the acceptance speech of Herbert Hoover, delivered August 11, 1928:

> "Moreover, the time has arrived when we must undertake a larger visioned development of our water resources. Every drop which runs to the sea without yielding its full economic service is a waste.
>
> "Nearly all of our greater drainages contain within themselves possibilities of cheapened transportation, irrigation, reclamation, domestic water supply, hydroelectric power and frequently the necessities of flood control. But this development of our waters requires more definite national policies in the systematic coordination of those different works upon each drainage area. We have wasted scores of millions by projects undertaken not as a part of a whole but as the consequence of purely local demands."

RELATION OF DIVERSION TO THE ECONOMIC USE OF STREAMS

The guiding principle of conservation of water resources is that a stream should be so developed as to serve in the greatest measure all beneficial purposes of which it is capable. This may involve diversion of a part of the flow to another drainage basin. If so, remuneration must be made in some manner such that the economic value as a whole within the drainage basin of the stream itself is not impaired. If this can be done and at the same time economic wealth added to another basin by the use of a part of the flow of the stream, the diversion of a part of the flow may be desirable. Thus there are evidently instances where diversion of a part of the waters of a stream should be included in a comprehensive plan of development, not only for public water supply but for a number of other uses. In fact, the larger diversions hitherto carried out have more often been for purposes other than municipal water supply. These include irrigation, of which there are numerous instances of important trans-watershed diversions. A less common purpose of diversion is for sewage dilution, as in the case of diversion from Lake Michigan by the Sanitary District of Chicago. Diversions on a small scale for navigation purposes in case of inland canals have been fairly common. New York state for a century, and until recently, diverted the runoff from a considerable area at the head-waters of the Susquehanna River into the Great Lakes drainage basin to supply the Erie Canal. It has made a simi-

lar diversion from the head-waters of the Hudson River into the Great Lakes drainage basin to supply the Champlain Canal. One of the uses served by the Chicago diversion has been for the benefit of open-channel navigation in the Illinois and Mississippi Rivers. Here the benefit, amounting at low stages to several feet increase in navigable depth in the Illinois River, has been at the expense of loss of navigable depth approaching 6 inches as a maximum limit in the harbors of Lakes Michigan and Huron, and a smaller loss of navigable depth in Lakes Erie and Ontario. In such instances if only one purpose was served by the diversion and this use was detrimental to a similar use of the stream within its own drainage basin, the diversion could only be justified, in the opinion of the author, if it could be shown that the increased benefits were so great that all injury or loss of benefit in the basin of the stream itself could be adequately compensated and still leave a large margin of increase in economic value to the whole region served both by the stream itself and by the diversion.

Diversion from one drainage basin to another for water power production has not been common except where, as in the case of the Chicago diversion and numerous diversions for irrigation, water power development was incidental to the diversion made for some other primary purpose. In many instances adjacent streams are naturally adapted to diversion from one to the other for power production, especially in conjunction with the development of storage.

DIVERSION FOR WATER SUPPLY

The argument is sometimes made, in effect, that water supply is a prime necessity and hence all other possible present or potential uses of a stream should yield to this paramount use. The argument is even carried to the point that streams likely to be needed in the future for public water supply purposes should be set aside or allocated exclusively to that use, thus forever inhibiting their development for any of the other beneficial uses which they may serve. The author does not hold with any such extreme doctrine. The present and probable future needs of public water supplies should, of course, be given due and adequate consideration in any program for the development and utilization of a stream. A reasonable reservation of the best available sources of water supply for a community is desirable. This should be measured in terms of requirements for decades rather than for centuries; nor should the fact be lost sight of that communities require industries as well as water for their maintenance, and in-

dustries in general require both power and water for their operation. Furthermore, many cities are so fortunately located that alternative sources of supply are available.

Hitherto it appears that attention has not sufficiently been directed to the possibility of using for water supply purposes streams which are well adapted to that use but poorly adapted to other economic uses. It is certainly poor economy to take a stream for water supply purposes to the exclusion of all other uses where the stream contributes or is capable of contributing largely to the economic wealth of the same community which is to be supplied with water, if another supply of water can be obtained at a reasonable increase in cost without the destruction of such economic value.

Before setting apart important streams for water supply purposes alone because of the possibility of their future need, consideration should be given to certain factors in relation to municipal growth which are sometimes overlooked. Many communities, such as mining and lumbering towns, are essentially ephemeral, but even disregarding these and considering what may be called "permanent communities" alone, their growth does not proceed forever in a geometrical progression. For short periods of time in youthful or rapidly growing communities, future population may, of course, be predicted on decade percentages of population increase. The life histories of organisms, the growth records of communities in the older parts of the world and various other factors indicate conclusively that the growth curve of a community is not a parabolic curve approaching infinity, as if invested with seven-league boots, but is a type of ogee curve approaching a maximum or limiting line asymptotically. These facts are expressed in Mitscherlich's law of growth, to which those who take delight in the prognostication of populations yet unborn may well give heed.

LEGAL ASPECTS OF DIVERSION

The method of diversion used in a particular case largely depends on (a) the economic aspects of the diversion and (b) the jurisdictional conditions and legal rights of the parties. The engineering and economic aspects of diversion are, therefore, closely related to certain legal questions, and some discussion of legal questions is necessary to a broad treatment of the subject.

This discussion will be confined to a few simple principles, largely axiomatic in their nature.

The author desires to state that he is not a lawyer and presents such limited discussion of legal matters as is here given in the hope that such a discussion may be helpful in solving the legal and jurisdictional problems which arise whenever an important stream diversion for any purpose whatever is undertaken. The author presumes to enter the domain of the law to this extent, because of the fact that he happens to have participated in more than one hundred cases involving questions of diversion and riparian rights in the higher courts of various states and in the United States Supreme Court.

Riparian rights have their origin in the ownership of the *ripus* or bank of a stream. They are appurtenant to and part of the adjoining real estate. They are, therefore, property rights, subject to absolute ownership or the right "to have and to hold."

Diversion in general involves an invasion of such property rights of lower riparian owners on the diverted stream and it may involve legal questions on the stream, if any, into which the water is diverted.

There is a familiar legal axiom: "*Aqua currit et debet currere ut currere solebat.*" Black's Law Dictionary translates this: "Water runs and ought to run as it has used to run." Put in another form, the right to the use of a running stream is as it flows in nature, and only as it flows. More broadly, and in the light of accepted legal decisions in the eastern United States, the riparian owner has the right to have a natural stream flow over his property in its natural regimen, undiminished in volume and unimpaired in quality. This is not, however, to be too strictly interpreted. It is subject to two other legal maxims:

1. The principle of "*de minimis*", in accordance with which the courts will not recognize trivial matters and will not, in general, grant a perpetual injunction in a case where there is no injury or only trivial injury involved.

The extent and nature, and the narrow limitations of what constitutes negligible injury to the rights of a riparian owner are clearly set forth in the following quotation from "Angell on Watercourses", 1854 (pp. 127–128):

". . . . It was laid down by Ch. J. Kent that neither the occasional increase of the velocity of the current, and of the *quantum* of water below, nor the insensible evaporation and decrease of water by dams, will amount to an actionable injury. To adopt his own language,—'Many such circumstances may be inevitable from the establishment of one dam above another upon the stream; the question in such cases would turn upon the *nature and extent*

of the injury.' As a general rule, the law, as a practical science, cannot take notice of 'melting lines, nice discriminations, and evanescent quantities." In this view, the increased evaporation of water produced by the exposure of a larger surface, by means of the erection of a dam, would not be a *deprivation* of the water in the technical sense in which it is used as denoting *diversion*; though in a loose and theoretical sense it might be."

2. This seems essentially the basis of another legal principle which is of the utmost importance in matters of diversion—the principle of "reasonable use." A riparian owner can make a consumptive use of so much of the water as is required and necessary on his own land for ordinary domestic and agricultural uses. He can locally divert the flow of the stream provided he returns the water within the limits of his own property and he cannot complain if his neighbor upstream makes similar uses of the stream within his rights as riparian owner, even though such uses may be detrimental in a slight degree to the lower riparian owner. This, as I see it, is the principle of equal use and involves all there really is to the much-talked-of doctrine of "equitable division" of streams.

It is to be borne in mind, however, that reasonable use is primarily use by a *bona fide* riparian owner. A city located in one drainage basin and seeking to divert water from another drainage basin for municipal purposes is not a *bona fide* riparian owner even though the city may have acquired lands bordering on the stream at the point of intended diversion. The right to real estate is inalienable in the sense that one who covets his neighbor's property cannot take it without his neighbor's consent, even though he offers to pay what it is worth. Under the paramount right of the sovereign, real estate can, however, be taken from individuals for public use if it lies within the jurisdiction of the sovereign state. Thus one state can authorize the diversion of water for public use from one drainage basin to another drainage basin, both lying within the same state, but not otherwise.

There is another point, the significance of which is, I think, not always recognized, at least by engineers, in dealing with diversion problems, and that is that two or more riparian owners may by agreement perform acts of diversion which no one of these persons could legally perform without the consent of the others. Such joint action being in and of itself legal does not establish a precedent for the performance of illegal acts of the same nature. This is, I think, the underlying principle involved where compensation in kind, as it is called, has been undertaken in eastern United States.

A lower riparian owner injured as a result of diversion, even where the diversion is made for a public use and where both parties are within the same legal jurisdiction, is entitled to be paid in money an amount equal to the diminution in the fair market value of his property and he cannot be compelled to accept flow release, reserve flow, compensation in kind or any other substitute for money damages. He can, however, accept, for example, a regulated flow or compensation in kind if he so elects.

The states of New Jersey and Pennsylvania could, by mutual agreement, each permit the other to divert water from the Delaware River south of Tri-State Rock and they could do this without let or hindrance on the part of the state of New York, but the state of New York cannot divert water from the Delaware River at any point in such a manner as to do substantial damage in either the state of Pennsylvania or the state of New Jersey without the consent of each of those states, and the fact that one of those states may have consented to diversion on the part of New York state does not in any way affect the right of the other state to require that such diversion, if carried out, shall be performed in a manner which will result in no substantial injury in that particular state.

THE DELAWARE RIVER CASE

There have been several recent adjudications by the United States Supreme Court relative to proposed major diversions involving interstate rights. These include, in the order of decision:

The Chicago Diversion Case.
The Connecticut River Case.
The Delaware River Case.

There has also been much discussion of the proposed diversion of Colorado River at Boulder Dam, to which the State of Arizona dissents.

The Delaware River Case is the latest and perhaps most important case decided. It does not appear that the decision has, however, always been correctly understood or interpreted in such newspaper and other accounts as have been published. This case serves well to illustrate the application of the legal maxims above given and also to illustrate various engineering and economic aspects of diversion problems.

Delaware River leaves New York state at Tri-State Rock, near

Port Jervis, at which point it is joined by the Neversink River, one of its head-water tributaries, and has a drainage area below the junction of the Neversink of 3,415 square miles. It flows southward, the thread of the stream forming the state boundary line between New Jersey and Pennsylvania, a distance of about 120 miles, to the head of tidewater at Trenton. Downstream from Trenton the river is a navigable tidal estuary.

New York City proposed to divert 600 m.g.d. from the head-waters of Delaware River in New York state. A proposed Tri-State compact relative to this diversion having failed of ratification in New Jersey and Pennsylvania, and New Jersey not having succeeded in obtaining better terms than those proposed in the compact, a bill of complaint was filed by New Jersey in the United States Supreme Court praying for an injunction to restrain New York City from carrying out her proposed plan of diversion.

New York City proposed to take the water from five drainage areas tributary to Delaware River, as follows, (7):[2]

STREAM	DAM AT	WATERSHED ABOVE DAMS, SQUARE MILES	DIVERSION, GALLONS PER DAY
Neversink	Curry	70	70,000,000
East Branch of Delaware	Downsille	370	370,000,000
Beaverkill	Beaverkill	60	60,000,000
Willowemoc	Parkston	60	60,000,000
Little Delaware	Delhi	50	40,000,000
Total			600,000,000

The lowest point of the proposed diversion is at a distance, measured along the stream, of 108 miles from Port Jervis, and 228 miles from Trenton. The diversion from those areas was to be undertaken progressively in three stages:

	m.g.d.
1. The Neversink	70
2. East Branch Delaware River	370
3. Beaverkill, Willowemoc and Little Delaware	160

New York City proposed to make the diversion in accordance with the terms of the proposed compact, i.e., to provide compensation in

[2] Numbers in parentheses in this and subsequent paragraph refer to pages in the Report of the Special Master in the Delaware River Case.

kind to the extent of releasing a uniform flow of 0.45 c.s.m. from the diverted areas during the months July to October, and during other months to release the natural runoff of the diversion areas if less than 0.45 c.s.m., but to divert the entire runoff if greater than 0.45 c.s.m. This is a composite plan of diversion involving, in part, absolute diversion, in part, diversion with reserve flow and, in part, compensation in kind.

In the trial of the case Pennsylvania, as intervenor, proposed a modified plan of reserve flow and compensation, described later. It was claimed by New York City at various times in the course of the trial and argument that:

1. The proposed diversion would not result in damages.
2. New York City had money enough to pay the damages.
3. New York City could take the water as a matter of right.

New York City also proposed, as shown by one of the exhibits offered in the case, an ultimate diversion of over 2,600 m.g.d. from the drainage basin of Delaware River in New York state. Besides acceding to New York City's proposed initial diversion of 600 m.g.d., Pennsylvania requested that a still larger diversion by Pennsylvania should be authorized. New Jersey alleged that diversion under either the New York or Pennsylvania plan would result in substantial damages to Delaware River in New Jersey with respect to the utility of the stream in connection with public water supply,—particularly that of the city of Trenton,—industrial use of water, sanitation, water power, agriculture, recreation, navigation, fisheries, and the oyster industry.

New Jersey proposed that in the event that a permanent injunction should not be granted, the diversion should be based on a simple plan of compensation in kind by stream flow regulation on a "fifty-fifty" basis, the regulated flow of the diversion areas involved to be equally divided between compensation flow to Delaware River and diversion for water supply.

It was shown that the entire Delaware River was capable of regulation by storage so as to provide a uniform flow of about 1.85 c.s.m. or 90 percent of the mean flow and that the head-water areas involved in New York City's diversion could be regulated to a still higher basis and that the basis of diversion and compensation proposed by New Jersey would provide New York City 1 c.s.m. or 746,000 gallons per square mile per day from the diversion areas and at the same time would provide a regulated compensation flow of 1 c.s.m. from the same areas.

The case was heard before Charles N. Burch of Memphis as Special Master. The Special Master found that the diversion as proposed by New York City would:

(a) Involve substantial damages to New Jersey and that the diversion could be permitted only to such an extent and in such a manner as to do no substantial damage in New Jersey (193):

". . . . I reach the conclusion and so find that the proposed draft of New York City of the equivalent of 600 million gallons daily is excessive and that this draft should be reduced to the point where it is clear that the amount permitted to be taken will not inflict substantial damage on New Jersey. . . . ."

(b) That the proposed third stage, involving the taking of 160 m.g.d. from the Beaverkill, Willowemoc and Little Delaware should be enjoined (195) and that the diversion should be limited to the taking of 440 m.g.d. (195).

(c) That in connection with the initial diversion of 70 m.g.d. from the Neversink, the Pennsylvania plan of release—compensation should be tried out, i.e., whenever the flow after diversion at Tri-State Rock or Trenton Falls falls below 0.50 c.s.m.

"or below 0.50 c.s.m. at both points, then, in that event, releases shall be made from the impounding reservoirs so as to restore the flow to 0.50 c.s.m., provided, however, the amount to be released does not exceed 30 per cent of the water yield of the diversion area, and as the water yield of the diversion area is 2.2 c.s.m., the maximum release (30 per cent) will be 0.66 c.s.m., and in a diversion area of 440 square miles (the completion of the second stage) a release of 0.66 c.s.m. multiplied by 440, gives a maximum release of 290.4 c.f.s." (196).

This is not a final but only a tentative method, to be subsequently modified as may be necessary to fulfill the fundamental requirement of the decision that there must be no substantial damage resulting from the diversion in the state of New Jersey. On this point certain paragraphs in the report of the Special Master are significant.

"Only 70 million gallons daily are to be diverted from the Neversink and this diversion will be in operation for a number of years before the larger diversion from the East Branch. During the operation of the Neversink diversion it can be determined with more accuracy than can now be forecast whether the Pennsylvania plan should remain as it is or whether it should be modified. Careful observations of the effect of the Neversink diversion will determine whether 0.50 c.s.m. is the proper control figure or whether that figure should be raised to 0.60 c.s.m., or higher. It can also be determine whether one, two or more control points should be adopted and the time of transit at varying stages of water can also be determined. With the result of this experience, the

quantity and plan of releases can be determined before the East Branch diversion has begun. The diversion from the Neversink is relatively so small that any factors of error in the Pennsylvania plan of release will have no materially adverse effect. I am therefore of the opinion that the decree fixing the plan of release should be made subject to modification." (196).

". . . . Any such change as may be necessary can be complied with by either increasing the storage capacity of the reservoirs which are contemplated to be built *or by building additional reservoirs on other tributaries of the Delaware for the purpose of storing water, which would be used only for release into the Delaware watershed.*" (198). (Italics are the author's).

The decision of the Supreme Court is in substantial accordance with the recommendations contained in the report of the Special Master. There are other points, such as the denial of the request of Pennsylvania for the allocation of water for its use, and the requirement for sewage treatment at Port Jervis, before diversion is undertaken.

The Special Master found in his report that the City of New York is not a riparian owner on the Delaware River or its tributaries (200); that

". . . . Delaware River constitutes navigable waters of the United States from Trenton northward to the junction of the East and West Branch at Hancock, New York and for an undefined distance above Hancock on the East Branch and the West Branch . . . ." (83)

and that

". . . . any decree rendered in this case is necessarily subject to the paramount authority of Congress over navigation, as well as the authority of the Secretary of War and Chief of Engineers, in pursuance of the powers granted to them. The United States is not a party to this suit and in no way bound by any decree which may be rendered. *New York City must take the risk of any action which Congress or the War Department may see fit to take affecting the carrying out of the plans of that city . . . .*" (94–95). (Italics are the author's.)

The significant fact is the failure of New York City to obtain either a definite or an unconditional right of diversion of any quantity of water whatever from Delaware River or its tributaries. Apparently the United States Supreme Court is inclined to construe the axiom of reasonsable use of a stream more broadly in cases involving adjoining states than in cases involving the rights of individuals only, but I find nothing in this decision construing the principle of equitable division of the waters of streams in the eastern United States in the manner in which Pennsylvania sought to have it construed. While the de-

cision apparently recognizes the existence of such a principle, the Special Master states (201):

"The equality of right of each state under the Constitution must be preserved and the principle of equitable apportionment fairly applied."

Apparently the Special Master undertook to apply this principle fairly by restricting diversion to substantially what could be done under the simple, common law axiom of reasonable use on the part of riparian owners, already referred to, but subject to certain compensation and reserve flow provisions, so that no substantial damage should result from the diversion, as regards the state of New Jersey. It is true that as the result of this decision it may be considered that the doctrine of diversion subject to compensation in kind as practiced in England has been imposed upon the state of New Jersey. This is hardly the true situation. New Jersey proposed in the course of the trial that, if an injunction should not be granted, and any diversion was to be permitted, then the diversion should be predicated on a fair basis of compensation in kind.

The author represented the state of New Jersey as Engineer-in-charge during the preparation and trial of this case and thinks he can say that the officials bound with the duty of protecting the interests of New Jersey were always willing to permit a reasonable and by no means inconsiderable amount of diversion of water for water supply purposes on the part of New York City, subject to a just and equitable basis of compensation in kind.

The case was tried during the great drought of 1930. This drought has now passed. Various economic changes have taken place. New York City has not undertaken the proposed diversion. In a large measure the matter is left hanging in the air and New York City seems to have been placed very much in the position of Shylock in "The Merchant of Venice" in that she can have her "pound of flesh" from the state of New Jersey but she must take it in such a manner as not to inflict any material damage to New Jersey's interests in the water resources of Delaware River. The author's present feeling is that a more definite and clean-cut decision on the basis of a fixed compensation flow of 1 c.s.m., with the right of New York City to divert an equal quantity of water per square mile, would have been preferable, at least to New Jersey.

DIVERSION EFFECTS

These may be important both with respect to (a) the stream from which the diversion is made, and which will be described below the point of diversion as the "diverted stream," and (b) the stream into which the water is diverted, which will be described as the "receiving stream."

Diversion may be undertaken in a great variety of ways with reference to the relations between the flow diverted, the yield of the area and the flow allowed to pass downstream after diversion. For convenience in discussion, the methods of diversion in this respect may be divided into three principal classes:

1. Absolute diversion,—where the entire yield of the drainage area above some point is taken and diverted, with the possible exception of waste during floods.

2. Diversion subject to reserve flow. In this instance the natural flow up to some fixed limit is allowed to pass downstream in its natural regimen.

3. Diversion subject to compensation in kind. In this instance some degree of regulated flow is substituted and passed downstream in place of the natural regimen of the stream.

The primary effect of diversion is to change the regimen of both the diverted stream and the receiving stream, both as to volume of flow and stream stages downstream, respectively, from the point of diversion of one and the point of inversion of the other. The magnitude of the diversion effects depends largely on the mode of diversion and the problem is complicated by tributary inflow to the streams below the points of diversion or inversion.

At a given cross-section on the stream the stage—discharge relation may usually be expressed in the form

$$q = kh^n,$$

where h is the stage or depth of stream above a point of zero discharge. With some exceptions, such as the St. Clair and Detroit Rivers, the exponent n is greater than unity, usually 1.5 to 2.5, and a given increment or decrement of discharge at a low stage produces much greater change of stream depth than is produced by the same increment at a higher stage of the stream. It follows that a constant volume of diversion taken out of the natural flow may seriously affect low-water stages at points downstream and impair the utility of the stream for navigation, boating, bathing and recreational uses without

materially benefiting the stream by reduction of flood stages. It is interesting to note that in case of a stream into which water is diverted, the lower stages of the stream are increased much more than the high stages. Thus the diversion of about 8,500 c.f.s. from Lake Michigan into the Illinois River by the Sanitary District of Chicago had the effect of increasing the low stages of the middle Illinois River from 3 to 5 feet, although it only lowered the level of Lakes Michigan-Huron by about 0.5 foot.

The effect of diversion on stream stages usually decreases proceeding downstream from the point of diversion. This is in part due to "delays in transit" and in part to variations in runoff conditions in different parts of the drainage basin. For example, in the case of the proposed Delaware River diversion, where a reserve flow of 0.45 c.s.m. was provided at the point of diversion and the flow to be diverted was taken out of natural runoff of the diverted areas in excess of this amount, the author's investigations showed that in the case of a heavy runoff resulting from local rains on the diverted area, the diversion of the excess above 0.45 c.s.m. would at times reduce the navigable depths at the head of the tidal section of the Delaware River at Trenton by amounts in excess of 0.5 foot, although Trenton is more than 200 miles downstream from the proposed point of diversion.

It is also true that the effect of the operation of a given plan or program of diversion becomes less certain and definite as the distance downstream increases. Thus a given reserve flow or compensation flow adequate at the point of diversion may provide water when it is not needed and fail to provide water when most needed at points remote from the point of diversion.

In case the reserve flow method is used it may be desirable to base the reserve flow on the actual flow of the remaining drainage area measured at some point remote from the point of diversion. For example, the release of reserve flow is predicated on the remaining natural flow at the cities of Port Jervis and Trenton in the United States Supreme Court decision in the Delaware River Case. Even this is but a makeshift remedy as several days may be required for reserve flow if released to reach a given point downstream, especially under winter conditions, and the matter is further complicated where there are intermediate ponds or regulating reservoirs. Separate discussion of the downstream effects of diversion on each of the different economic uses of a stream cannot be undertaken here. The effect on navigation, recreational uses, and agriculture, is closely re-

lated to the changes in stream stages resulting from the diversion. Water power has been singled out for somewhat detailed discussion in relation to diversion effects because of—

a) Its economic importance.

b) The fact that it is the most common basis of diversion damages.

c) It affords an excellent illustration of the economic benefits of storage regulation.

EFFECT OF DIVERSION ON WATER POWER

There is a not uncommon misunderstanding in regard to the present status and economics of water power. The assertion is not infrequently made, in effect, that with large, modern, high-pressure boilers and high economy steam turbines, steam power can be produced more cheaply than water power. It is possible that such statements have been disseminated for propagandistic purposes, sometimes perhaps by those representing water works systems desiring to divert water from streams, but no doubt also by power companies desiring to acquire riparian rights and power sites. The idea is an old one and underlies Solomon's proverb: "It is naught, it is naught, saith the buyer, but when he is gone his way then he boasteth" of the bargain he has made.

On the other hand, some steadying influence has undoubtedly been needed to counteract the effect of sentimental conservationism that prevailed with regard to water resources more or less during the earlier years of the present century. The basis of the misunderstanding is, however, due probably to failure to take into account certain engineering and economic factors involved in the production cost both of steam and water power, particularly the effect of load factor or varying load demands. Hydro power is generally used for public utility service and the public utility must supply its customers with power on demand, not merely with base or firm power. In a system having a maximum demand of 100,000 kw., perhaps 20,000 or 25,000 kw. will be firm or base load, while the maximum demand will be two or three times the average load and four or five times as great as the base load. The peak demands are of relatively short duration and represent but a small portion of the total energy requirement, yet capacity must be provided and maintained at all times adequate to supply the highest demands.

The cost of a steam plant is closely proportional to its capacity, so that the investment and overhead charges necessary to supply, say, the highest 20 percent of demand, are substantially the same

per kilowatt as the investment and overhead charges necessary on the plant which supplies the base load. The figures in table 1 are from actual records and show the energy requirement and duration of various percentages of the load demand in a medium sized, eastern, electric utility system. Thus the highest 20 percent of total demand has a duration of but 2½ percent of the time or 0.6 of an hour per day, and the energy required to supply this demand represents only 2 per-

TABLE 1

Duration and energy requirements for peak load demands

PERCENT OF DEMAND	AVERAGE DAILY LOAD DURATION, PERCENT OF TIME	PERCENT OF ENERGY OUTPUT
Highest 10	0.5	0.4
Highest 20	2.5	2.0
Highest 30	12.5	5.0
Highest 40	26.5	11.0
Highest 50	43.0	20.0
Highest 60	59.0	31.0
Highest 70	75.0	45.0

TABLE 2

Illustration of cost of peak energy in a steam plant capable of producing base load energy (100 percent load factor) at 0.50 cents per kilowatt-hour

PERCENT OF CAPACITY DEMANDED	COST, CENTS PER KW.-HR.
Highest 10	11.25
Highest 20	6.50
Highest 30	3.61
Highest 40	2.17
Highest 50	1.58
100	0.50

cent of the total energy requirement of the system, yet it requires an installed capacity of 20 percent of the total for the system. Not only must capacity be provided to supply these demand peaks, but if the energy is produced by steam, coal is consumed in banking boiler fires during idle hours, which may represent the greater portion of the time.

Table 2 shows the cost of energy production in a modern steam plant to supply peak demands representing various percentages of the

total demand. These figures are, of course, illustrative only, as the cost of steam installation, like the cost of water power development, varies throughout a wide range, depending on the conditions. The figures are, however, derived from data given in the Report of the Northeast Super Power Committee under the direction of the Secretary of Commerce (now President), the Honorable Herbert Hoover, in 1924.[3] They have been modified to the basis of a plant which could produce steam power at a cost of ½ cent per kw.-hr. if operated on a 100 percent load factor basis. It will be noted that in such a steam plant the production cost for a small part of the output represented by the highest 10, 20 or 30 percent of the demand may be several cents per kilowatt-hour.

As contrasted with run-of-the-stream hydro plants, common in the earliest years of this century, the modern tendency in hydro-electric power development involves the use of a stream regulated by storage and a power plant supplied by adequate pondage so that it can be utilized mainly or even exclusively for the production of power to meet peak load demands.

A large part of the installation cost of a hydro-electric power plant, including that for dams, reservoirs and real estate, is fixed for a given location and the cost of increased capacity is represented by the cost of additional turbines, power house, penstock and electrical equipment. These combined items may, for example, represent one-fourth to one-half of the total investment cost per kilowatt of capacity, so that increase in capacity or ability to carry peak loads in a hydro plant can in general be obtained at a much lower cost per kilowatt than in a steam plant.

The increased cost of producing peak energy in such a hydro plant is represented chiefly by the overhead charges on the additional items named. When the plant is idle, the overhead charges go on just as in the case of the steam plant, but there is no expenditure for coal for banking fires. It may happen that a hydro plant which could not produce base or firm power economically in competition with a steam plant may be able to produce peak power at much less than its cost of production by steam, and it would be foolish to use such a hydro plant for base power production. The comparison of the energy production costs in hydro and in steam plants on the basis of firm power alone may therefore be misleading and meaningless.

[3] Super Power Studies, Northeast Section of United States, Federal Power Commission, May, 1924.

Referring again to the tables and using, for illustration, hydroelectric production costs in a large eastern utility system using hydro power to meet peak load demands, where the production cost for hydro power for the highest 40 percent of the load demand is of the order of 0.9 cent per kw.-hr., the same energy if produced in an economical steam plant would cost around 2 cents per kw.-hr. It should be noted that attention here is directed to the cost of that part of the energy represented by peak demands above a certain percentage of the total demand.

Failure to recognize the economic relations of steam and hydro power under conditions such as described has apparently arisen in part from the general practice of dealing exclusively in average power production costs. Owing to the fact that the energy involved in meeting peak demands represents but a small part of the total energy requirement, the difference in production cost as between base load and the peak energy requirement does not stand out clearly in figures representing average costs alone.

If the stream to be diverted is lacking in adequate natural reservoir sites, storage regulation may be unfeasible and it may happen in such a case that diversion with an adequate reserved flow may not materially impair the water power resources of the stream. If, on the other hand, extensive regulation of the stream is commercially feasible in conjunction with power development, a major diversion may materially reduce the regulated flow available for power, with consequent reduction in the economic resources of the stream. Conditions where streams are regulated by storage and fully developed and utilized mainly for energy production for peak demands are by no means uncommon or extreme. This is true of several of the larger rivers in New York state, including Mongaup River, one of the tributaries of Delaware River.

For a fully regulated stream the reduction in available power as a result of diversion will be more or less proportionate to the fraction of the total runoff which is diverted, since all the flow diverted comes out of flow that would produce power.

If the diversion is either absolute or is made on the reserve flow basis, a commercially feasible power project may, as a result of the reduced output available following diversion, become commercially unfeasible and claims for damages are likely to be correspondingly larger.

For a stream fully capable of regulation by storage, the loss in available power output is the same for the same diversion whether the flow

release method or compensation in kind is utilized. It does not, however, follow that the effect on power will be the same, since compensation in kind provides regulation which is beneficial to power and with a proper basis of such compensation a considerable part of the investment otherwise necessary for power development with storage may be obviated and the rate of return on the remaining power available remain unimpaired.

HYDRAULIC PROBLEMS OF DIVERSION

Diversion involves interesting and sometimes unusual hydraulic problems, both as to effect of the diversion on the diverted stream below the point of diversion and as to effect on the stream into which the water is diverted. Usually the object of diversion is to provide a definite flow, either uniform or regulated seasonally. The flow diverted may be regulated either antecedent or subsequent to the diversion. The diversion from Esopus Creek through the Ashokan reservoir to New York City represents an instance of the diversion of a nearly uniform flow regulated antecedent to the diversion. The diversion of Basic Creek into Alcove reservoir in connection with the new water supply system of the city of Albany, N. Y. represents an instance of the diversion of a varying natural flow regulated subsequent to the diversion.

For antecedent regulation the first problem which arises is to find the requisite storage or reservoir capacity to provide the desired uniform, minimum, regulated flow. This is a familiar problem.

If the regulation is subsequent to diversion, the problem arises of determining the tunnel or diversion channel capacity required to divert the given total volume of flow from the natural runoff of the stream. The total annual volume of diversion which can be made with a given conduit capacity is equal to the area of the stream flow duration curve below a horizontal line corresponding to the conduit capacity.

Diversion by pondage

Another type of conditions is involved in what may appropriately be called "diversion by pondage." In applying this method a dam is constructed downstream from the entrance to the diversion tunnel. As long as the stream flow does not exceed the tunnel capacity, all of the flow is diverted. If the inflow to the pond exceeds the tunnel capacity the pond begins to fill. When the pond is once filled, any

excess is wasted but diversion proceeds at full tunnel capacity as long as the pond remains full. When inflow decreases, the pond is drawn through the tunnel. Thus for every major rise in stream flow one pondful of water is diverted in addition to the water which could be diverted through the same tunnel without the pond.

On a flashy stream a diversion pond may greatly increase the volume of diversion with a given conduit capacity or, conversely, it may materially reduce the size and cost of tunnel needed to provide a given total annual volume of diversion.

Such diversion intake ponds are well adapted for irrigation and water power diversions where the final regulation is made in the drainage basin into which the water is diverted. As an example, in the case of Lochaber water power scheme in Scotland, the major part of the runoff from eleven flashy mountain streams is diverted through a common tunnel into a regulating reservoir by means of diversion intakes and ponds each of limited capacity.[4]

In connection with water supply an example of a diversion of this type is afforded by the Basic Creek dam and tunnel near Albany. The pond is not used for storage but is emptied after every storm. The amount of diversion with pondage in a given instance cannot be determined directly from the flow—duration curve or by the ordinary method of application of a mass diagram, since the diversion is made essentially storm by storm. The problem is complicated by the fact that for any particular period of runoff three conditions may arise:

1. The intake pond may only partially fill.
2. The intake pond may fill and waste occur.
3. The pond may be partially filled at the beginning of a rise in the stream.

The divertible volume equals the total runoff less the waste, and waste occurs only at critical periods of high runoff. The solution of the problem is simplified by determining the waste, since this involves computations only for relatively short intervals. No computation is needed for periods of normal and low flow when, as can be seen from an ordinary hydrograph, no waste will occur.

An orderly procedure for the solution of this problem in the simplest manner that will give accurate results was developed by the author in connection with the Basic Creek diversion and is described in an appendix.

[4] Halcrow, W. T., The Lochaber Water Power Scheme; Proc. Inst. C. E., 23: 1930–31.

Effect of inversion on floods

An important problem arising in connection with a stream into which water is diverted at a uniform rate is to determine the effect of the diverted water on maximum or flood stages of the receiving stream.

For convenience, the water diverted into the stream may be designated "canal water." The increase in stage at a given cross-section due to the presence of canal water is precisely equal to the difference between the stage due to the total flow past the given section and the stage due to the natural flow alone, or to the total flow minus the canal flow passing the given cross-section. As long as the regimen is uniform the rate of canal outflow past a given cross-section will be equal to the rate of canal inflow to the stream. During the passage of a flood the flow past a given cross-section is not uniform. As the flood stage increases, the ratio of the natural flow to the canal inflow increases and, owing to pondage effects, the proportion of diverted water or canal water in the flow past a given cross-section may not be the same as the ratio of the diversion rate to the natural inflow rate to the stream.

The analytical treatment of this problem is by no means simple. It can, however, be shown that for a constant rate of canal inflow to the drainage basin the quantity of canal flow passing a given cross-section is less than the rate of canal inflow to the drainage basin while the stream is rising and until after it has passed its crest stage.[5] The

[5] This conclusion is based on the analysis of a typical case where, as is generally though perhaps not always true in natural rivers, the volume S of storage in the stream channel varies as a higher power of the stage h than does the discharge rate q at the outflow section. For all such cases the ratio $\frac{q}{S}$ decreases as the stage increases.

After the author's paper was written and after reviewing the author's analysis, Mr. Jacob A. Harman, M. Am. Soc. C. E., made a further study showing that for this condition, i.e., for $\frac{q}{S}$ decreasing with rising stages, the rate of canal outflow at the flood crest is less than the rate of canal inflow, and flood stages are not increased by the full amount of the inversion. If $\frac{q}{S}$ is constant, then the increase in flood crest stage corresponds to the increase in inflow, while if a case can occur where $\frac{q}{S}$ increases as the stream stage increases, the flood crest stage may be increased slightly more than the increment of stage due to the constant canal inflow.

canal outflow then becomes precisely equal to the canal inflow and during the remaining receding stages the canal flow past a given cross-section is greater than the canal inflow or diversion into the stream. Finally, the total volume of canal flow passing a given cross-section from beginning to end of a flood rise and subsequent recession is precisely equal to the canal inflow to the stream during the same period. The important result is that diversion into a drainage basin at a constant rate, say 10,000 c.f.s., does not increase the discharge at a given location on the receiving stream at the time of a flood crest by as much as 10,000 c.f.s. and the increment of flood stage is also increased by a less amount than that due to an increment of 10,000 c.f.s. at the given location.

Time of transit

Not only in connection with problems of diversion, particularly where reserve flow is provided, but in general in connection with stream regulation by storage reservoirs, questions of time of transit of water between different points along the stream frequently arise. Time of transit or the time at which the effect of a change in regimen of a stream at one point will be felt at a point farther downstream may be widely different in different cases, even for the stream at the same mean stage throughout the reach in question. Suppose that a stream is flowing with a uniform, steady regimen. Then the time of transit is obviously the actual time required for water to be transported throughout the reach under consideration, that is, it equals the length of the reach divided by the mean velocity.

Consider a case of diversion at a given point "A" on a stream, the diversion being subject to some degree of reserve flow. If a rise of the stream occurs such that there is excess flow which is diverted, it seems obvious that in some degree, at least, the effect of this diversion will actually be felt at a point "B", some distance downstream, at the time at which the residual or reserve flow arrives at "B", since this is the flow which arrives at "B" in place of the natural flow. This will be after the effect of the general rise in the stream has subsided and the stream has returned to a more or less steady regimen.

Viewed from another angle, it may also appear that the time at which the effect of the given diversion will be felt at the point "B" is the time at which the natural rise in the stream above point "A" would have arrived at "B" if part of the rise had not been diverted.

This interval may be considerably shorter than the actual time required for the reduced flow to travel from "A" to "B".

Neither viewpoint is entirely correct. It is often assumed—as was, in fact, assumed by Pennsylvania in the Delaware River Case—that the time of transit is the difference of time of occurrence at "A" and "B" of crests of natural rises in the stream due to rainfall. This assumption may be approximately true for very great rises due to prolonged rains over the entire drainage basin. A little consideration will show that it is grossly in error in case of small and medium rises and that the differences in time of occurrence of crests of such rises has little or no relation to and is much less than the actual time of transit of water from point to point, or the time required for the effect of the suppression of the rise above point "A" to reach point "B" on the stream.

Suppose the rain producing the rise in a stream is of short duration, much less than the so-called "time of concentration" or time required for runoff to reach the point "B" from all parts of the drainage basin. The crest of the rise at any point "X" along the stream will then occur precisely at the time at which the outflow from the local reach of the stream in which the point "X" is located has risen to a volume equal to the inflow to the same reach.

This crest time for any local reach will, in general, occur very soon after rain stops, since the land inflow to the stream then rapidly decreases—in fact, the crests will occur almost immediately after the cessation of rain at the head of the drainage basin but will be somewhat delayed at points downstream. Exceptions occur in case of streams with flood plains and for unusual rain distribution, and in exceptional cases the crest of a rise may occur at an earlier time near the mouth of the stream than near its source. If the rain is general and ends simultaneously over the whole drainage basin the difference of time of crest occurrence for a short rain may be a matter of hours at points so separated as to require days for the water actually to travel the same distance.

There is a simple experiment which demonstrates clearly that difference in time of crest occurrence of rises in streams has nothing to do with time of transit of diversion or inversion effects. If at a given instant water begins to flow into a long trough laterally throughout its length, the water will instantly begin to rise from end to end of the trough. If, now, some suitable coloring matter is put in the upper end of the trough and the lateral inflow is quickly stopped, it will be found

that the crest rise due to the inflow has passed any given point along the trough some time before the coloring matter arrives at the same point. In the Delaware River Case New Jersey relied upon measurements of the actual time of travel of floats and the observed time of travel of diurnal waves of water released from power plants. The latter gives the time of transit of a crest superimposed on a more or less uniform regimen. This, again, is not the same as the time of transit for a decrement of flow due to diversion.

Since there are so many apparent answers to this problem, either approximate or, as in the case of the flood crest time, wholly incorrect, the question may naturally be asked: How can the time of transit of reserve flows or diversion effects be determined? Determination of the actual time of travel of the water for uniform flow is the correct starting point. Beyond this, variable flow is involved, and problems of variable flow can only be stated in terms of differential equations for which exact solutions are not available. The component of time of transit due to variable flow in a natural river channel is superposed on the ordinary velocity of actual travel of the water under steady flow conditions. This added component may be either positive or negative according as it is the result of an increment or decrement of flow at the point of diversion.

Space does not permit discussion of experimental data or methods of computation applicable in such cases. There are, however, various experimental data available, including the classical but little known experiments of Darcy and Bazin, and there are various theoretical and some empirical formulas for idealized cases.

This question of time of transit has been presented at some length for the reasons that:

1. There is little on the subject in engineering literature.
2. It is the most difficult hydraulic problem arising in connection with diversion.
3. It serves to illustrate the difficulty of preserving an undisturbed regimen or even providing a system of flows equal to the natural regimen by any simple system of reserve flow or flow released at a point of diversion.

ADJUSTMENT OF DIVERSION TO ECONOMIC CONDITIONS

Diversion of a stream, however beneficial it may be to the divertor, is in general inimical or detrimental to other beneficial uses of the stream within its own drainage basin. This is especially true if di-

version is undertaken without regulation. Stream flow regulation by storage is in general beneficial to all uses, though there may be some minor exceptions, and regulation may permit diversion to be made in such a manner as to eliminate or at least minimize injury to other uses of the stream. Excluding diversion, the uses of a stream within its own drainage basin are in general more or less harmonious or at least are not vitally antagonistic. Thus the development of power in conjunction with storage regulation may materially benefit recreational uses, fisheries and navigation.

In the following paragraphs various methods of diversion are described and compared both with respect to their advantages or disadvantages as regards the divertor and with respect to the effect of the various methods of diversion on the economic utilization of the stream for purposes other than diversion.

(a) *A constant or nearly constant diversion* out of the base or steady flow. The diversion out of the Great Lakes by the Sanitary District of Chicago and many smaller diversions by municipalities for water supply purposes are of this type. This type of diversion leaves the lower riparian owners bereft of all or a part of the steady or natural minimum flow of the stream, which, in general, is the most valuable part, and, in general, flood flows are not appreciably diminished.

(b) *Diversion of low and normal flows*, allowing waste to take place into the diverted stream only at times of floods or excess flows. This is a common condition in connection with larger water supply diversions and irrigation diversions. The diverted stream loses all the more beneficial runoff from the diverted area. It is left below the point of diversion with a channel larger than required for the remaining flow at ordinary stages and which consequently may need flushing in low water. This type of diversion usually requires storage and the flood or excess flow may be much reduced below the natural flood discharge rate from the diverted area at points immediately downstream from the point of diversion. The time of occurrence of flood crests passing downstream from the diverted area is generally delayed as compared with the time of their occurrence for the natural stream. While, in general, downstream flood effects are reduced, the retarded crest of a flood from the diverted area may, as pointed out by Graeff many years ago, be brought into coincidence with crests from unregulated tributaries in such a manner as to accentuate the flood crests in the lower reaches of the stream.

(c) *Diversion with reserve flow.* This is practically the same as the preceding case except that when the natural runoff from the diversion area falls below a certain fixed limit, a quantity of flow equal to the natural runoff from the diverted area is released. In theory there is no change in the regimen or volume of flow below the point of diversion when the natural flow is less than the reserve flow provided for, but for rates of runoff exceeding the fixed reserve flow, the volume of discharge and stream stages below the point of diversion are materially reduced.

When the natural flow exceeds the reserve flow the divertor is permitted either to divert or to impound the excess. Since a constant, continuous diversion is usually required, storage is generally necessary, but it is not to be understood that this type of diversion benefits the stream through regulation. It provides no regulation whatever of the flow below the point of diversion—the regulation redounds wholly to the benefit of the divertor.

The first example of this type of diversion which has come to the author's attention was in connection with the Dolge grant for water supply of the city of Little Falls, N. Y. There the owner of extensive water power rights on East Canada Creek entered into an agreement with the city of Little Falls whereby the city was permitted to divert water from Spruce Creek, a tributary of East Canada Creek, for water supply purposes, subject to the requirement—

". . . . that it will at all times maintain the normal and natural flow of water (exclusive of storm water) in Spruce Creek below the point, where party of the second part, shall take water therefrom, for the purpose aforesaid, it being the intention of the parties hereto, that party of the second part, shall replace and return to the channel of said stream below the point of such taking, an amount of water, from the waters stored, above that point, on said Spruce Creek, and its tributaries, at least equal to the amount diverted by it, for the purpose of affording such public supply of water, to the City of Little Falls and its inhabitants."

This has been interpreted not as providing compensation in kind but as a reserve flow agreement, the intention being that the natural regimen of the stream shall remain unchanged except as to storm waters. Although the Dolge grant was made more than thirty-five years ago, the indefiniteness of its terms, particularly as to what constitutes "natural and normal flow" of the diversion stream has led to disputes and the matter has long been and still is in litigation. It was apparently the intent of the grant that the water diverted should be

from flows in excess of those which could usefully be applied to hydraulic power development and that, therefore, the diversion would not result in actual damage to water power.

Diversion with reserve flows was apparently first developed and applied on a definite basis by the author in the case of the diversion agreements entered into between a company engaged in supplying the city of Utica, N. Y., with water, and the various power owners on West Canada Creek, a tributary of Mohawk River, from which the water supply of Utica is mainly derived. The first of these agreements was entered into in 1905. The original agreement provided that the water company could divert water from, and only from, natural flow above the point of diversion in excess of power requirements for the then existing or any future plant. In that particular instance the point of diversion and the intake to the power plant involved were closely adjacent. At that time the existing power development had capacity to use 333 c.f.s. from a drainage area of 375 square miles. In order to make its diversion continuous at a uniform rate, the water company was obliged to provide and did provide storage within the drainage basin sufficient to supply the water it required from previously impounded excess water at times when the natural flow of the stream was less than the requirements of the power plant. There did not appear to be any likelihood of increase in the capacity of this power plant and to make the matter more definite it was agreed between the parties and incorporated in a supplemental agreement that the water company should be permitted to make its diversion directly from the stream or to impound water whenever the the remaining flow in the stream equalled or exceeded 335 c.f.s. Other diversion agreements between the same water company and power owners farther downstream were entered into either with the same limitation or with a limitation in the flow usable for power fixed by legal or physical conditions.

At the time of the Dolge grant and at the time the first Utica diversion agreements were entered into, water power development was generally carried out on the "run-of-the-stream" basis, using auxiliary steam plants if necessary to maintain a continuous power output. When the Trenton Falls power development was undertaken in 1900 it was impossible to obtain designs in the United States for turbines under so high a head as here required—265 feet—and low-speed, outward-flow or Fourneyron turbines of Swiss design were utilized. The installed capacity of 333 c.f.s., or

about 40 per cent of the mean flow of the stream, was looked upon as about the limit of practicable if not of economic capacity for this plant. As a result of subsequent regulation of the stream and changes in the art of power development, the capacity of the Trenton Falls power plant has, in fact, been increased to 1,700 c.f.s. The original diversion limit is a matter of contract between the power company and the water company and still stands, with the result that a diversion of 50 m.g.d., the total amount contemplated, will reduce the actual output of this plant by roundly 1,800 h.p. or over 10 million kilowatt-hours per year.

This example serves to show that diversion subject to flow release, while it might result in no injury to water power developed under the older conditions, is not likely to be acceptable nor to avoid serious injury to water power developed under modern conditions which call for regulation of stream flow by storage and the operation of the power plant with pondage in such a manner that the highest part of the load demand is provided by hydro power.

An advantage of the flow release plan of diversion from the diverter's viewpoint is that it may require only a relatively small volume of storage to permit a given continuous diversion. In the West Canada Creek case, 1,117 million gallons of storage has sufficed for 14 m.g.d. of continuous diversion.

In addition to its failure to eliminate damage in some cases, such as that above described, diversion by the reserve flow method is subject to practical difficulties of operation. In general all the inflow to the diversion reservoir must be measured. This is a difficult matter if, as is often the case, the diversion reservoir is fed directly by numerous tributaries. Furthermore, if the reservoir has a large area, the water yield of the reservoir area, which is represented by the difference between rainfall and evaporation, requires determination. As the result of uncertainty as to the precise rate of inflow it is difficult to determine whether or not a proper quantity of flow is being released to the stream in critical cases.

In the case of the proposed diversion from Delaware River drainage basin by the city of New York, a reserve flow of 0.45 c.s.m. was proposed for certain months of the year with, however, the provision that if the natural flow exceeded 0.45 c.s.m. the entire yield of the diversion area could be taken. Under these conditions if the runoff from the diversion area happened to be 0.44 c.s.m. this quantity of flow would be released, whereas if it happened to be 0.46 c.s.m. no

flow was to be released from the diverted area. In view of the difficulties of precisely determining the inflow to the diversion reservoir it is exceedingly difficult in the operation of this method for a lower riparian owner to determine for himself the manner in which the diversion agreement is being carried out. He must rely in a large measure on good faith of the divertor.

(d) *Compensation in kind.* This is the usual English practice in connection with water works diversion. The stream is regulated so as to provide at least a certain amount of continuous flow below the point of diversion. In England the amount of compensation flow to be provided is usually fixed by Parliament after a consideration of all of the uses and requirements of the stream, and the regulated, constant or compensation flow so provided is, as a rule, several times as great as the natural minimum flow of the stream.[6] Compensation in kind, therefore, provides some direct benefit to lower riparian owners. Where the method of diversion by compensation in kind is used, the quantity of flow passing downstream from the point of diversion is fixed and definite and a downstream riparian owner can at any time readily determine whether the diversion requirements are being fulfilled or not by simply reading a gage or inspecting a suitable measuring device installed at the point of diversion. It is, therefore, superior to the method of flow increase in simplicity and definiteness. If diversion is carried out by either the flow release method or the method of compensation in kind, there is, of course, a reduction in the volume of runoff of the stream below the point of diversion equal to the total volume of water diverted.

On many streams the greater portion of the total annual volume of runoff occurs during floods or smaller rises of the stream occasioned by storms. Where adequate natural reservoir sites exist and power is commercially valuable, it may be possible to regulate even such a stream in such a manner as to provide a uniform flow approaching more or less closely the mean flow. In such a case not only is the flood flow valuable, but since it provides the larger part of the volume of water which may be impounded, and hence provides the greater part of the regulated flow, flood waters instead of being a menace may become the most valuable part of the runoff from the drainage basin.

Where diversion rights subject to reserve flow are acquired by agree-

[6] Data for various examples of compensation in kind are appended to this paper.

ment between the parties, some monetary consideration is usually required in addition to the provision of reserved flow. Compensation in kind, if on an equitable, economic basis, should obviate or minimize such requirements.

As compared with taking of absolute rights of diversion by direct payment of damages, their acquisition in conjunction wth provisions for either reserved flow or compensation in kind possesses the important advantage to the divertor in that these methods permit, in effect, deferred payment. Particularly in case of compensation in kind, the payment is chiefly made by the provision of storage for diversion or compensation in kind, or both, and the storage need only be provided for, and to the extent of, the concurrent diversion. The advantage of initially acquiring rights adequate for future requirements for a reasonable period is obvious. The direct acquisition of diversion rights which will not be needed in full for some time in the future may involve considerable investment not presently remunerative.

The available alternatives are either acquisition of rights on a reserved flow or compensation basis, on the one hand, or, on the other hand, the acquisition of absolute diversion rights on a piece-meal basis as needed to meet increased demands. The latter method is likely to be costly to the divertor and inimical to the best economic utilization of the stream, since in that case lower riparian owners are never certain as to the extent to which the available drainage area may be reduced by successive encroachments for diversion. In any event, it is evident that major diversions should only be carried out after the formulation and adoption, if possible, of a general plan or program of utilization of the stream, with due regard to all the available kinds of service it may render, and participated in by all of the parties interested.

Appendix 1

Computation of Diversion by Pondage

It is assumed that the diversion is made by means of a tunnel or conduit located upstream from a dam on the diverted stream, the diversion conduit having its invert at the bottom of the pond or at a fixed level below the spillway. The conduit is always open and the operation is automatic. During floods the pond fills up; diversion takes place at the maximum rate when the pond is full to or above spillway level, and as the inflow recedes a volume of water equal to

the capacity of the pond is diverted and the pond thereafter remains nearly empty, ready to catch the next succeeding rise in the stream.

The problem is to determine the proportion of the total flow which can be diverted. It is assumed that a record of the natural flow of

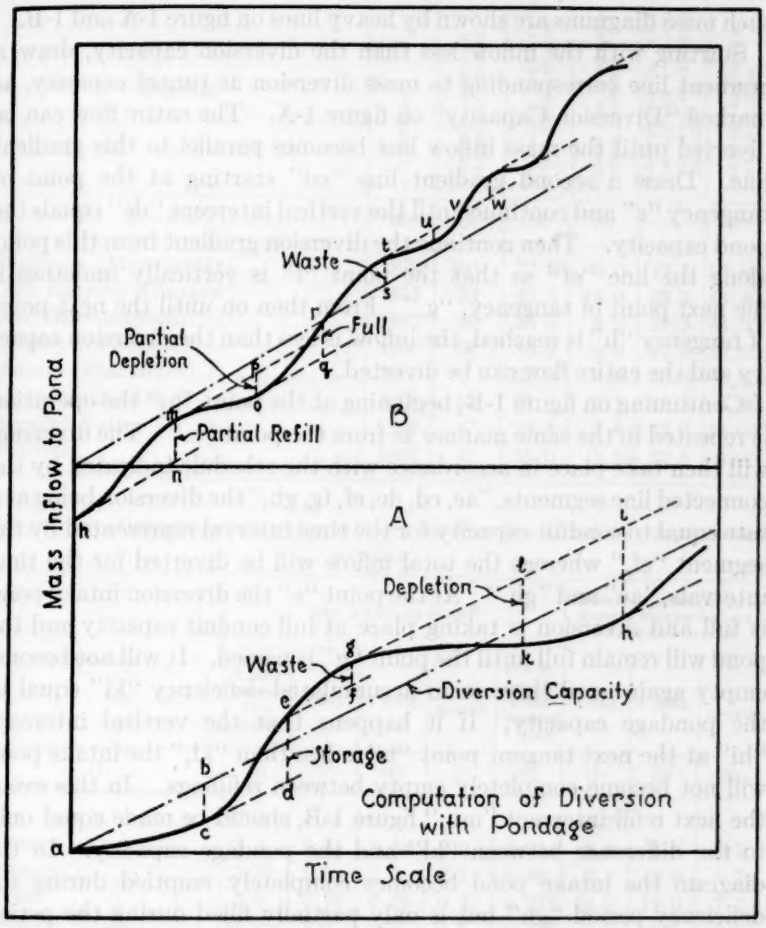

FIG. 1

the stream is available at the point of diversion. The method of procedure consists in determining the quantity of water wasted over the spillway and subtracting this from the total flow to obtain the net amount available for diversion. During periods

of low flow or flow less than conduit capacity, the entire flow can be diverted. Such periods can readily be eliminated from the daily flow records or hydrographs. Mass diagrams of daily flow should then be plotted for the remaining periods, during which the flow is above or alternately above and below conduit capacity. Segments of such mass diagrams are shown by heavy lines on figure 1-A and 1-B.

Starting with the inflow less than the diversion capacity, draw a gradient line corresponding to mass diversion at tunnel capacity, as marked "Diversion Capacity" on figure 1-A. The entire flow can be diverted until the mass inflow line becomes parallel to this gradient line. Draw a second gradient line "cd" starting at the point of tangency "c" and continue until the vertical intercept "de" equals the pond capacity. Then continue the diversion gradient from this point along the line "ef" so that the point "f" is vertically underneath the next point of tangency, "g." From then on until the next point of tangency "h" is reached, the inflow is less than the diversion capacity and the entire flow can be diverted.

Continuing on figure 1-B; beginning at the point "h," the operation is repeated in the same manner as from the point "c." The diversion will then take place in accordance with the schedule indicated by the connected line segments, "ac, cd, de, ef, fg, gh," the diversion being at a rate equal to conduit capacity for the time interval represented by the segment "ef," whereas the total inflow will be diverted for the time intervals, "ac" and "gh." At the point "e" the diversion intake pond is full and diversion is taking place at full conduit capacity and the pond will remain full until the point "g" is passed. It will not become empty again until there is an accumulated deficiency "kl" equal to the pondage capacity. If it happens that the vertical intercept "hi" at the next tangent point "h" is less than "kl," the intake pond will not become completely empty between refillings. In this event the next refill intercept "mn," figure 1-B, should be made equal only to the difference between "kl" and the pondage capacity. In the diagram the intake pond becomes completely emptied during the deficiency period "gh" but is only partially filled during the period "mn," and only partly emptied during the period "op."

Using this method, the first object is to determine the waste. This is the sum of the vertical waste intercepts and is readily scaled from the diagram. The waste deducted from the total inflow gives the diversion.

APPENDIX 2

EXAMPLES OF COMPENSATION IN KIND

Data as to compensation water in England

LOCATION	AUTHORIZED	ASSUMED AVERAGE RAINFALL, INCHES	AVAILABLE YIELD IN 3 DRY YEARS, INCHES	WATER FOR COMPENSATION, INCHES	PROPORTION OF COMPENSATION WATER TO AVAILABLE YIELD, PERCENT	AREA OF DRAINAGE BASIN, ACRES
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Thirlmere (Manchester)	1879	100	72.0	9.0	12.5	10,800
Loch Katrine (Glasgow)	1855	74	45.2	15.0	33.3	46,400
Elan (Birmingham)	1892	69	36.0	9.9	27.5	44,000
Vyrnwy (Liverpool)	1880	61	36.9	9.9	27.0	22,000
Dunsop (Blackburn)	1877		27.0	9.0	33.3	6,800
Alwen (Birkenhead)	1907	52	27.6	9.2	33.3	6,300
Derwent Valley	1899	47	23.4	7.8	33.3	32,000
Weardale	1866	30	10.0	3.25	32.5	6,000

Comparison of compensation water with dry weather flow (English compensation practice)

LOCATION	DRY WEATHER FLOW[a]					PROPORTION OF COMPENSATION WATER TO DRY WEATHER FLOW OF		
	Compensation water	1 day	1 week	1 month	C.F.S. per 100 acres basis of 1 week	1 day	1 week	1 month
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Elan	27.0	4.0				6.75		
Vyrnwy	13.5	2.4	2.54	3.9	0.214	5.6	5.3	3.5
Dunsop	3.8[b]	1.8	1.9	2.2	0.59	2.1	2.0	1.7
Alwen	3.6	0.19	0.26	0.47	0.076	19.0	14.0	7.7
Derwent Valley	15.2	5.0	6.0	7.6	0.348	3.0	2.5	2.0
Weardale	1.22		0.16		0.05		7.5	

[a] Columns (2) (3) (4) and (5) are in units of millions of British Imperial gallons per day.

[b] Corporation gave money payment in respect of 2½ m.g.d., reducing this quantity to 1.35 m.g.

In English practice compensation water is usually impounded in, and released from, the storage reservoirs from which the supply for diversion is obtained and is constant in amount. American practice usually differs in that compensation reservoirs are often separate from and on different branches of the stream from the diversion reservoirs. The outflow from the compensation reservoir may or may not be constant but the compensation water is usually let down at the will of the riparian owners.

Black River, N. Y. In 1849 the state of New York constructed the Boonville feeder to supply the summit level of the Erie Canal with water derived from Black River at Forestport. Chap. 181 of the Laws of 1851 provided (ref. 1) for the construction of "reservoirs of sufficient capacity to supply the Black River Canal feeder with such quantity of water during the summer months as shall be necessary for the supply of Black River and Erie Canals and as shall give to the Black River, as near as may be, as much water as ordinarily flows therein during the summer months." In accordance with this act reservoirs were constructed on Black River and its tributaries having capacity of 3,860,853,000 cubic feet on tributary drainage areas aggregating 461.8 sq. mi. This storage is adequate to provide a regulated flow from the given area and with the heavy rainfall of the region, equivalent to about 500,000 gals. per sq. mi., or a total of 231 mil. gals. or 358 c.f.s. constant outflow. Of this an average of 205 c.f.s. is diverted outside the drainage basin for canal uses,[7] leaving 153 c.f.s. regulated compensation flow. The compensation flow is equivalent to 43 percent of the total regulated supply. Applying this rate to Delaware River conditions with a possible regulated supply of 1 mill. galls. per sq. mi. per day, the corresponding rate of compensation would be 0.65 c.s.m.

Little Falls, N. Y. In the case of one of the larger power users affected by the diversion of water from Beaver Brook and Spruce Creek for the supply of Little Falls, N.Y., a compensation agreement was entered into about 1901 providing for the maintenance of a minimum compensation flow of 9.36 c.f.s. as against a maximum possible diversion of four million gallons per 24 hours. In this instance the compensation flow was one and one-half times the diverted flow. The area tributary to the compensation reservoir is 36.2 sq. mi., while that tributary to the main diversion reservoir is 5.1 sq. mi.

[7] During the navigation season, May-November, inclusive.

New York State Storage Law. Chap. 762, Laws of 1915, State of New York providing for river regulation by storage reservoirs, contains the following:

Sec. 10—The words 'average normal flow' when used in this article shall be construed to mean that result in terms of cubic feet of water per second derived by the division of the total number of cubic feet of water flowed by any stream or river, at the location thereon where the measurement thereof is taken, on those days in a year during which such flow did not exceed the average flow, by the number of seconds in the total number of such days.

Part VIII

470. Flow of river. When the flow of a river at any water power-plant thereon falls below the average normal flow thereof, the then natural flow of the river at a reservoir dam thereon shall not be restricted without the consent of the owner of such power-plant."

On the basis of these definitions the average normal flow of the Delaware River or its tributaries amounts to something over *0.80 c.s.m.* and this represents the reserve flow required to be maintained on this basis in accordance with the New York water storage law.

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DISCUSSION

THOMAS MADDOCK:[8] Many engineers and lawyers apparently are of the opinion that the decisions of the United States Supreme Court in the Connecticut and Delaware River cases have clarified water law. Some writers say a "new law" has been constituted regarding the division of the water and power of interstate streams.

Engineering opinion influences the making of laws relating to water. The inter-dependence that exists between lawyers and engi-

[8] Engineer, Colorado River Commission of Arizona, Tuscon, Ariz.

neers, who work together on water problems, justifies the engineer in inquiring into this subject.

If a "new law" has been created, by the court it must have been done to supply an omission or to supplant an old law. If the "new law" is reasonable it will survive. If not, it will in turn be supplanted, when found to be unreasonable. A reasonable law should be just, its meaning should be plain, and it should be capable of general application.

If ancient laws and customs, and values created by them, are to be abandoned, in an effort to secure something better, it would be well to consider the fundamentals involved, and compare new and old laws and decisions to judge their respectvie merit.

The "Constitution" on "water"

The authority for the "new law" is obscure. The constitution of the United States is silent regarding the use of water for irrigation or hydroelectric purposes. The framers of the Constitution were unacquainted with these matters. It should be obvious that the Constitution did not delegate or imply the grant of power to control matters beyond the knowledge of its framers. On the other hand, no language can be plainer or broader than the Tenth Amendment "the powers not delegated to the United States by the Constitution, nor prohibited by it to the states are reserved to the states respectively or to the people." This amendment limits the judicial as well as the two other branches of Government.

The Constitution provides a method for its amendment, to permit its adjustment to changing conditions or opinions and to supply omissions. Many amendements have been proposed. Nineteen have been adopted. The Constitution recognizes that certain powers were not given to the Federal Government, nor could they be limited to the jurisdiction of the individual states, and provides how compacts or agreements can be made between states to control such matters. Therefore, constitutionally there should be no hiatus or conflict of authority, or necessity for ultra constitutional decisions.

The three branches of our Federal Government were created by the Constitution. None of these possess powers not given to them by it, but the absence of specific delegation of water control to any branch of our government has lead four different agencies to try to assume this power. Today the Executive, Legislative and Judicial Branches of the Federal Government, and also the State Governments,

are trying to control and distribute the benefits to be derived from using water for domestic, irrigation and power purposes. Naturally each of these human agencies consider themselves logically and legally entitled to assume such control; but as their ambitions conflict numerous controversies arise.

Bureaucratic competition

In the Executive Branch, the War Department, the Federal Power Commission and the Departments of Interior and Agriculture, through the Reclamation, Forest, Indian and Park Services, are trying to control water. The oldest of these agencies, the War Department, has been the most successful, as the result of Federal laws, giving it jurisdiction under the constitutional power conferred by article 1, Section 8, Paragraph 3 "Congress shall have power to regulate commerce with foreign nations, and among the several states and Indian tribes." The power here given has been rather liberally construed by court decision to imply the improvement of rivers, harbors, lakes and even to their artificial creation. This construction suggests unlimited control of water by the War Department.

The Federal Power Commission has asserted its jurisdiction over the power on navigable rivers and beyond to the innumerable small tributaries of such streams, also to the waters on the Federal Public Domain and Reservations. They claim to control 85 percent of the water power in the United States.

The Reclamation Service has contended in court that the Federal Government had the sole power over water, and that they were the agency to control its distribution for irrigation purposes, and also any power incidentally developed by irrigation works.

The Federal Power Commission and Reclamation Service so far has been handicapped in establishing absolute water control because of provisions in the Acts creating them, that the consent of the states was necessary. The desire to secure Federal funds has heretofore insured quick state approval for Reclamation projects. The present tendency for the Reclamation Service to become a mere Federal distributor of funds derived exclusively from the Western States may cause opposition to develop. The Reclamation Service has been able to control the division of some interstate waters by its power to select the projects to be first constructed on interstate streams.

The Park, Forest and Indian Services are limited in authority to the domain they supervise, in which they claim to be supreme, but

they have added millions of acres to their controlled areas in the last few years, and legislation is pending that would give them control of many millions more acres. These three Bureaus tend to add a Federal sovereignty, independent of state sovereignty to the Federal proprietary power they now possess. They also seek independence of Congressional appropriations by developing local sources of income. Some of the Western States are objecting to Federal Reservations being extended to include water power sites, and their exemption from state taxation.

The Indian Service contends that the present Indian population is entitled to sufficient water for its needs, and in addition that a future increased Indian population could recapture whatever water it needed at the expense of other water users. The Indians, by multiplying, could regain the entire West if this theory is accepted.

Congressional control of water

Direct Congressional control and division of interstate waters was attempted by Congress in the Boulder Dam Act by the approval of a Six State Compact dividing the waters of seven states between two artificially created areas. Congress also executed a contract with California, in which that state agreed to limit its use of water in exchange for an appropriation of $165,000,000.00, for its development.

Congress, or the Departments created by it, could thus control water, if the law of prior appropriation governs by supplying or denying funds for competing projects, and by granting or denying rights of way. At the present time the Secretary of Interior is executing contracts for the delivery of water and power from the Colorado River to various California Cities and irrigation projects, regardless of navigation needs or the "new law" of "equitable division."

State control of water

State officials have long contended that, except for navigation purposes the control of water was a state prerogative reserved by the Constitution. The Eastern States have considered that their water was controlled by the riparian doctrine of the common law, where not superseded by state statutes. In the West the state laws generally follow the appropriation doctrine.

The constitutions of many states assert state ownership and control of waters. Others presume ownership and specify how the right to use water may be obtained from the state. The Federal Legisla-

tive and Executive Departments approved the provisions of the States' Constitution, claiming ownership of water when these states were admitted into the Union. The Federal Judicial Department, until lately, has recognized the portions of the states' laws and constitutions in regard to the control of water, and has followed these state laws in deciding interstate water cases, where the laws of the states interested were similar.

Supreme Court control of water

Recently the Supreme Court has assumed the right to directly control the divisions of interstate water. Their decisions termed a "new law" have been called the law of "equitable division" or apportionment. The guiding principles of this new law have been summarized as follows:

"1. The doctrine of equitable apportionment controls the division and use of interstate waters.
2. Priority of appropriation creates no superiority of right in interstate waters.
3. The highest use of interstate waters is for drinking and other domestic purposes.
4. Injunctive relief will be granted only upon proof of present and substantial damage.
5. Diversion will be allowed from one watershed to another.
6. States may select sources of water supply provided no substantial damage is done to sister riparian states.
7. Release of compensation water depends upon down stream river conditions and needs.
8. Judicial allowance of diversion affecting navigation or navigable capacity is conditioned upon Federal approval."

These principles nullify both the riparian law of the East and the appropriation law of the West. They constitute such a depature from past decisions as to be revolutionary. Apparently they set aside the theories of both the states and the Federal agencies that have been contending for the control of water, and place full power in the Supreme Court to divide interstate waters in each case, as it considers best, without any general rule of procedure. The temporary nature of these late decisions, and their dependence on changing conditions preclude water users of an assurance of future use.

The conflict of authority over water needed to be settled, but it is probable that these eight principles instead of clarifying the subject make water rights more chaotic.

Principles 1 and 2 conflict with the provision of the law of prior appropriations enunciated in the Wyoming vs. Colorado case, U. S. 259.

Principle 3 conflicts with Principle 8 because domestic use of water can not be the highest if it must yield to navigation demands. It would require a new "equitable division" each time that cities in one state grew faster than the other cities or rural district in another state.

Principle 4 conflicts with the theory of "equitable division" itself as what damages or injunctions should one state secure against another, in its use of its allotted equitable apportionment of water.

Principle 5 is superfluous, and constitutes an unnecessary interference with the laws of different states, six of which permit, while eighteen prohibit, diversion from one watershed to another. It would seem that "equitable division" once made between states they should be permitted to control their apportioned water as they choose.

Principle 6 is ambiguous. May one state invade another, select and build diversion works without complying with the laws of the state invaded? May a city in one state construct water works in another and have them exempt from taxation, while it permits one of its own counties to tax the water works of a city in another of its counties, as Inyo County California taxes the present Los Angeles Aqueduct? Water can be a treasure without being subject to a law of "finders keepers."

Principle 7 raises the question, why should one state have to give compensating water or incur any outside expense in return for the use of its own equitable apportionment of a river?

Principle 8, makes a decision of the Supreme Court regarding an equitable division of water subordinate to the opinion of the War Department as to possible injury to navigation. Can any engineer who has considered the various clearance heights above navigable streams imposed upon bridge builders by the War Department anticipate what diversion from a navigable stream would meet with its approval?

Perhaps the engineers who have written about the new law have not exactly expressed the ideas of the court. Perhaps the court may later enlarge on its findings. Certainly it appeared reluctant to definitely settle the matters involved, but retained jurisdiction in these interstate cases to permit of future considerations, if and when necessary. The court's action invites discussion.

The words "equitable division" or apportionment are not descrip-

tive of a law, but are similar to a court saying it would render justice by being just.

Dissimilar decisions

An investigation of the various court decisions on interstate law shows, not only the absence of any general rule that could be termed a law, but also that the court has never made a final definite equitable division of an interstate stream. It has but temporarily permitted a state to use a portion of interstate water.

In the Kansas-Colorado case, U. S. 206, the court decided that Colorado was taking no more than its equitable share of the Arkansas River, but did not state the amount of such share. Therefore, each new use of water in Colorado and reduction in Kansas would require a law suit, instead of the engineers and lawyers knowing the amount of water apportioned to each state before funds were wasted on construction.

In the Delaware case, New Jersey vs. New York, U. S. 282, the Master and the court generalized about the "new law" of "equitable division," but the decision was not based upon it. Instead, the New York diversion was reduced to preclude damage to New Jersey. The court decided that the proposed diversion of 600 m.g.d. by New York would cause "more than slight damage" to the navigation and oyster industry of New Jersey. It reduced this amount to 440 m.g.d., and refused New York a firm right to this amount by prohibiting it becoming a prior appropriation. It provided New York City must construct reservoirs form which to release compensating water, and put in works to purify the sewage at Port Jervis. It retained jurisdiction of the case, so that any actual (rather than anticipated) damages could later be enjoined or adjusted. These items have so increased the cost and liabilities that New York City will probably abandon Delaware diversion. "Equitable division" in this decision, therefore, appears surplusage, and the older law of "where there is no damage there can be no recovery" takes its place.

In the Delaware case the court said "the master adopted the principle of "equitable division," which clearly results from the decisions of the last quarter of a century." This is debatable. The principle case in the West during this time was Wyoming vs. Colorado, U. S. 259. It was decided in 1922, after eleven years of litigation. "Equitable division" was not said to control this case; instead the court decreed that prior appropriations should govern the use of water in both

states, regardless of state lines because this law was in force in each state. After announcing this principle, the court departed from it, (just as they departed from "equitable division" in the Delaware case) and required the senior appropriators to make the additional investments to secure their water supply against the taking of 15,500 acre feet of water per year by junior appropriators. A logical conclusion of the law of prior appropriations would require a junior, rather than a senior appropriator to pay for storing additional water. This case was recently reopened, which indicates it is unsettled after twenty-two years.

Equity

"Equitable division" is undoubtedly what the court is striving to secure, but if minor navigation can disregard equitable division for domestic use, should not the reserved constitutional provisions of state ownership and control also nullify "equitable division?" In other words, can this "new law" supplant but a portion of our old laws or Constitution? Should not the Supreme Court review the order of the War Department if it may review the actions of Congress or the States?

An immutable supremacy for navigation would be uneconomic and itself inequitable. If navigation is paramount, as the court decided in the Arizona-California case, can Arizona or its citizens, prohibit the lowering of the Boulder Reservoir for domestic, irrigation and power purposes during drought periods by insisting on the right to navigate the reservoir for some 120 miles?

Can Arizona enjoin the construction of the Imperial and Parker Dams or additional depletion of the water of the Colorado River by the Upper Basin, Los Angeles or Coachella Valley in order to retain continuous navigation on the 350 miles of river below the Boulder Dam? Can the court dismiss Arizona's case without trial or consideration of "equitable division" in the name of navigation, and then later destroy navigation by imposing a new law of "equitable division" or, after the Court has legalized the Boulder Dam Act, because of navigation, can the War Department permit reservoir depletion and stream diversion because of lack of navigation?

In the Connecticut case the Master considered the relative purity of the various sources of supply and the disinclination of people to accept an inferior water if better is obtainable. If this factor is considered then the "new law" of equitable division depends, in part, on

the relative pollution of streams that the states permit, and logically those states which are most negligent in protecting their streams will have the better right to interstate waters. As Massachusetts permitted New Hampshire to pollute the Merimac, her equitable apportionment of the Connecticut was increased.

If the "new law" of "equitable division" includes the right to take water provided damages are paid then "equitable division" partly depends on the wealth of the state seeking to take water, and the earlier a state was settled, or the richer it is, the greater would be its equitable share of interstate waters.

Impractical equitable division

The doctrine of "equitable division," contrary to riparian law or prior appropriations, is extremely complicated. It involves: the relative rights of cities to grow beyond the limits of their natural resources by diverting the natural resources of other sections of the country; whether it is better for the future American citizens to be concentrated within a few large cities or distributed among many medium sized ones; whether people should move to resources or resources should be moved to where people have perhaps mistakenly congregated.

Equitable apportionment of waters in the West, where the irrigable lands greatly exceed the available water supply, should include consideration of so many factors, most of which are subject to change, as to be impossible of human attainment. The area of the land in each state capable of irrigation, the benefit to be obtained by using an acre foot of water in various sections, the cost and probable return from the reclamation of various districts, the national necessity for semi-tropic products against those produced in the temperature zone, the total area of each state in the watershed, the amount of other waters a state possesses for its development, the other sources of power like coal and oil available, the amount of water each state contributes to a stream, the population benefitted in each state or which might be sustained by irrigation of the various areas, the relative fall of the river in each state that can be used to create power, the lands that must be destroyed in one state to provide storage reservoirs, and the relative value of water for navigation-domestic, irrigation, power and recreational purposes are a few of the factors that must be considered if the division of interstate waters is to be by equity rather than law, interstate compacts or constitutional amendments.

Judicial inconsistencies

The court prohibited the diversion of 4 percent of the water of the Great Lakes by Chicago for sanitary domestic purposes. The court allowed the 5 per cent diversion of water from the Laramie in the Wyoming-Colorado case for irrigation purposes. the court authorized Boston's proposal to divert 2 percent of the Connecticut River. The court refused to allow New York to divert 24 percent of the water of the Delaware originating in two streams in New York, but permitted 17 percent diversion subject to damages and compensations for the lower riparian owners. The court refused to consider the threatened diversion of 5½ percent of the Colorado for the City of Los Angeles, and refused to pass on the proposed diversion of 41 percent of the main Colorado Rivers, California is being assisted to take by the Federal appropriations from a stream said by the court to be navigable.

In the Delaware case the court said, "Before this court can be moved to exercise its extraordinary power under the Constitution to control the conduct of one state at the suit of another the threatened invasion of rights must be of serious magnitude, and it must be established by clear and convincing evidence." The court saw such evidence and magnitude on the Connecticut and Delaware, but not on the Colorado.

The threatened diversion by Boston and New York, large as they seem, are but fractions of the threatened diversions from the Colorado River, which Congress appropriated funds to pay for. Before the court dismissed Arizona's suit the government had made surveys; part of the Colorado River work was contracted; the Secretary of Interior had executed contracts for the delivery of the water to be stored and for the sale of the power to be produced, yet in dismissing the Arizona suit the court said: "When the Bill was *filed* the construction of the dam and reservoir had not been commenced. Years must elapse before the project is completed. . . . . The bill is dismissed without prejudice to an application for relief in case the stored water is used in such a way as to interfere with Arizona's rights." How did the threat of California diversion differ from that of Massachusetts and New York, except in being much greater?

In the Connecticut case the court found in regard to the 50,000 hydroelectric power installation authorized by the Federal Power Commission at Kings Island, which might be adversely affected by the Massachusetts diversion, "This work, so far as the testimony

shows, has not been resolved on by the company, and as far as shown the required capital has not been subscribed or arranged for." Contrast this with the Colorado River case, where the court refused to review contracts for the sale of power on which the Boulder Dam construction depends, which were impossible of fulfillment, unless among other things the Metropolitan Water District voted $220,000,000 of bonds. This occurred four months after the court's decision.

In the Delaware case the Master refused to consider, as now pertinent that Philadelphia and Eastern Pennsylvania would need 750 m.g.d. by 1980. By a coincidence this same date of 1980 is the optomistic estimated time when Los Angeles will need the additional 1,000 m.g.d. from the Colorado River, and the power to lift it 1600 feet over the cost mountain range.

Economic uncertainty

From an economic standpoint the dismissal of the Arizona-California case is regrettable. The amount of water and power in dispute are greater than in any case ever presented to the court. The future location of some 15 billion dollars of new wealth, and the relative right of the states, congress and the federal bureaus are involved. The appropriation of $165,000,000 was contingent upon valid contracts being executed to repay the Federal Government, and the success or failure of the Los Angeles aqueduct to cost approximately $300,000,000 is dependent on the right of the Metropolitan District to Colorado River water and power. If it cannot have the power it cannot pump the water, or if it cannot have the water it cannot buy the power, and thus pay for the dam. If Arizona and Nevada may impose a sales tax on electrical output the whole project may be a financial failure. Yet the court dismissed this case without passing on these water and power questions. In Kansas vs. Colorado the court said that "the judicial powers are general and include whatever is not denied," also "we should settle questions that neither Congress nor the states can." In dismissing Arizona's suit the court did not settle many vital questions.

In the Kansas-Colorado case the court recognized the right of the states to control water and rejected the plea of the Federal Reclamation Service attorneys that the Federal Government owned the waters. It stated that "Congress has no general grant of power;" but "that Congress' powers were specified in the Constitution;" "that no one of the enumerated powers by implication refers to the

reclamation of land;" "that it may be we have no power adequate to reclaim lands," "but if so, it should be granted by Constitutional amendment." In the Wyoming-Colorado case the court completely ignored a demand for federal water control.

Legal fiction

In Arizona vs. California the court rejected Arizona's plea that the Boulder Dam Act was for irrigation, domestic and power purposes rather than for navigation, and said, "But a court may take judicial notice that a river is navigable." During most of the last 20 years the Colorado has had no channel to the sea. The Federal Government authorized six immovable bridges crossing the river, constructed a dam without locks at Laguna, permitted another at Hanlon Heading that precludes navigation, and no locks are included in the plans for the Boulder Dam. There has been no commerce on the river for over 20 years.

From a practical standpoint navigation is a legal fiction. In the Child Labor case U. S. 259 the court said, "All others can see and understand this. How can we properly shut our minds to it?" "It is our duty to decline to enforce laws dealing with subjects not entrusted to congress but left by the supreme law to the states." "We cannot avoid this duty even tho it requires us to refuse to give effect to legislation designed to promote the highest good." In McCullough vs. Maryland, Chief Justice Marshall said; "Should Congress under pretext of executing its power pass laws for the accomplishing of objects not entrusted to the Government it would become the painful duty of this tribunal to say such an act was not the law of the land." But in Arizona vs. California the court said, "Into the motives which induced members of Congress to enact the Boulder Dam Act this court may not inquire." If the Supreme Court meant it can not inquire of the intent of a law, Congress could validate any law by declaring it to be for a proper purpose, and destroy all Constitutional safeguards.

Speaking of the necessity of the intent being in accord with the text of a law in the Child Labor case, the court said, "The proof of the contrary is found on the very face of its provision." The same is true of the Boulder Dam Act.

When passing the Boulder Dam Act, Congress approved the Colorado River Compact. The former said the Act was "for the purpose of controlling floods, improving navigation, regulating the flow and storing the water for reclamation, etc." On the contrary the lat-

ter said, "Inasmuch as the Colorado River has ceased to be navigable .... the use of its waters for the purpose of navigation shall be subservient to domestic, agricultural and power purposes." The court held that the former language controlled, as it was a specific provision, while the latter was general. If these two opposite statements, (which indicate how carelessly Congress enacted this measure) stood alone the latter would appear to a layman to be the more definite and specific, but the latter provision is strengthened in the Act by the language. "The U. S. .... etc. shall be controlled by the Colorado River Compact in the operation of said reservoir, canals, etc. anything in this Act to the contrary notwithstanding, etc."

The further intent of Congress to subordinate navigation is shown by the fact that it approved an agreement, allotting 7,500,000 acre feet consumptive use of water to the Upper Basin, including parts of Colorado, New Mexico, Utah and Wyoming, etc., and made a contract with California permitting that state to take 4,400,000 acre feet of the apportioned water plus one half of the surplus, or 3,100,000, or a total of 7,500,000 acre feet, which use would destroy all prospect of navigation, to say nothing of the court's statement that the Boulder Dam Act leaves Arizona's "legal rights unimpaired," for Arizona alone, if unrestrained, could use enough water from the Colorado River to preclude any navigation on its lower reaches.

Indirect unconstitutionality

But Arizona may be restrained. The court in its decision said, "Arizona has, of course, no legal right to use, in aid of appropriation, any land of the United States, and it cannot complain of the provisions conditioning the use of such public lands."

Perhaps this is dictum. If not, the Federal Government by original ownership, purchase or exchange of public lands as a proprietary owner, (if not subject to eminent domain) can prohibit the construction of dams, canals, tunnels, transmission lines, roads, railroads, pipe, sewer, telephone, telegraph lines, etc. It follows that if control of water is in the states (as the court said in the Kansas-Colorado case) and Congress may not directly usurp this power it may do so indirectly. This can occur in the East by purchase as well as in the West by original federal ownership, on the St. Lawrence, Potomac, etc. as on the Colorado or Columbia.

This is not an academic question. Congress ignored the Supreme Court's decision that the court alone had the authority to settle dis-

putes between states, and attempted to divide the waters of the Colorado River between two basins in the Boulder Dam Act. Congress approved of an allotment of 7,500,000 acre feet to the Upper Basin, and contracted with California to take 7,500,000 acre feet of water; the latter without regard to evaporation losses, navigation needs or possible award of water by treaty to Mexico. The allotment to California was made regardless of the fact that Congress approved a compact permitting the use of only 8,500,000 acre feet in Arizona, California, Nevada and parts of Utah and New Mexico. About 2,500,000 acre feet is already being used in Arizona, New Mexico, Nevada and Utah, so that Congress authorized California to take more than all the Lower Basin states are permitted to use of new water under the Santa Fe compact. When California, with the aid of funds from the federal government, uses all the water contracted to her, Nevada, New Mexico and Utah (which states approved the Colorado compact) can make no more development in the lower basin until 1963.

Alienating resources

The Boulder Dam Act provided "privilege for the generation of hydro-electric energy shall be given first to a state and the states of Arizona, California and Nevada shall be given equal opportunity as such applicants." The state of Nevada applied for one third the power. It was allotted 18 percent of the firm power. The States of Arizona and California did not apply. California power users applied and were allotted 64 percent of the firm, and all of the cheap dump power. The Attorney General of the United States, when asked by the Secretary of Interior for his opinion of the above part of the Act ruled in substance that the Boulder Dam Act made the states equal as *applicants* rather than *recipients* of power.

The Supreme Court might divide the benefits to come from Boulder power equitably between Arizona and Nevada that contain the dam site, but it is hard to see how it could equitably ever approve the administration's program of allotting two-thirds of the power to the distant State of California.

Bureaucratic favoritism

The Secretary of Interior expended Federal funds on a survey of All American Canal despite the Boulder Dam Act, which said no funds should be expended until the states took certain actions. He

was instructed by this same act to report before December 10, 1931 on Arizona irrigation projects. This was not done. Apparently the Executive Departments ignores the Legislative branch of the Federal Government, Congress ignores the court, and the court will not review their action.

The Federal Power Commission rejected applications of both the State of Arizona and private companies for Colorado River projects at other sites than Boulder. Special federal commissions refused a request to investigate these other sites at the expense of Arizona. The court has intimated that its opinion of equitable division is subordinate to that of the Secretary of War, who supervises commerce.

The Secretary of Interior controls the public lands. With the consent of the President he has recently withdrawn millions of acres in Arizona and the other western states, and purchased or exchanged other lands for the stated purpose of creating National Parks and Indian Reservations, over which he has complete control. These withdrawals and existing reservations cover practically all the water power sites in Arizona.

If these Federal agencies may restrain Arizona in her natural competition under the doctrine of prior appropriation, previously approved by all three branches of the Federal Government, while Congress expends hundreds of millions of dollars to advance the development of California's cities and farms, and nationalizes or confiscates Arizona and Nevada hydro-electric power to pay for this California development, and if the Supreme Court refuses to try a suit of Arizona seeking to enjoin such actions can it say that "Arizona's legal rights are unimpaired?" Are moral rights to natural resources and constitutional rights of state equality with other states capable of being destroyed without a state's legal rights being impaired?

The Court versus The Court

Instead of a similarity in the court's decision and the adoption of a law of "equitable division" a study of the various cases indicate the following:

Kansas-Colorado—U. S. 206. The citizens of one state may take water previously in use in another, but not (an indefinite) too much. Small damages are not compensable. The States, not the nation, own the waters. The Supreme Court, not Congress, can divide interstate waters.

Wyoming-Colorado—U. S. 259. Where both states are governed

by the appropriation law whoever uses water first is entitled to continue such use, but may have to expend an additional indefinite amount for storage to insure water for later users in another state.

Wisconsin-Illinois—U. S. 278 and 281. Compensating works to protect navigation will not justify diversion.

Domestic use in Chicago may not encroach on lakes' navigation by diverting 4 percent of their waters; by contrast a contract between the Secretary of Interior and Los Angeles to divert 5½ percent of the navigable Colorado is not a thread of damage sufficient for court adjudication.

Conn.-Mass.—U. S. 282 and New Jersey vs. New York, U. S. 283. The Riparian and Appropriation laws are abrogated. Threatened diversion is worthy of court's adjudication. Each state is entitled to use, for an indefinite time, an indefinite equitable apportionment of interstate streams provided its use does not immediately materially damage the other state's use, or impair navigation. Compensating works to protect navigation are required.

Child Labor—Case—U. S. 259. This court will judge the intent and effect of Congressional Acts.

Ariz. vs. Calif.—U. S. 283. This court will not examine the intent of an Act of Congress, declaring dams and power plants are being constructed for navigation purposes, nor try any other issue as to the respective rights of the states to the use of water for domestic, irrigation and power purposes, until after hundreds of millions of dollars have been expended by the states, cities, Federal Government and power companies, and an indefinite amount of work has been done to destroy navigation, establish water priorities, etc. Congress can control water indirectly, if not directly, through United States ownership of public lands.

CONCLUSIONS

Large districts always seek to become larger by absorbing the natural resources of others. Districts possessing economic advantages try to retain them for their own development. Majorities are as ruthless as conquering invaders, and minorities need constitutional safeguards, especially if in becoming states in a nation they surrender the right of armed defence.

Special legislation may secure a majority support that would be

absent from similar general legislation. Both the Legislative and Executive branches of the Federal Government are subject to a political pressure that gives a large voting district an advantage in the passing or enforcing of special legislation.

Our appointed courts were not intended to legislate or formulate matters of policy. The courts dislike to oppose Executive programs, or to decide Congressional acts unconstitutional. States will support general laws, but are as resentful and suspicious of special decisions as of special legislation.

Unless states possess sovereignty there will be no interstate compacts. No state will submit to the restrictions of compacts if it can secure better terms from Federal officials.

Heretofore the states believed that they controlled their water. Apparently the courts believe otherwise, but if the riparian and appropriation doctrine have been abrogated without a new law being constituted, passed or decreed there is now no general rule or law regarding interstate water. A multitude of suits will ensue. An enormous amount of public and private funds will be wasted, and much injustice will be done to small states unless a definite, general interstate water law is secured.

H. K. BARROWS:[9] An interesting phase of this subject is found where the headwaters of a stream may, due to a combination of natural and artificial conditions, flow into either of two drainage basins. An illustration of this on a considerable scale is found in the adjacent headwaters of the Allegash and Penobscot Rivers in Maine. This includes Telos, Chamberlain and Allegash Lakes with a total drainage area of 270 square miles, which years ago drained into Allegash River and thence to the St. John River in Canada. Dams now control these lakes, however, so that flow is into the Penobscot basin. This situation has been one requiring international negotiations and is still under consideration.

The speaker has very fully covered the essential phases of this subject and it remains only to accentuate some of them.

The legal aspects are clearly presented and practically resolve to the proposition that a *bona fide* riparian owner can not be deprived of water to his detriment without recourse. The practical adjustment of diversion damages, however, often tends toward compensation in kind, which brings in many interesting hydraulic problems.

[9] Consulting Hydraulic Engineer, Professor Hydraulic Engineering, Massachesetts Institute of Technology, Boston, Mass.

A taking of water entirely away from a river definitely and adversely affects that river—the water taken will never be returned and the only palliative, aside from damages paid, lies in readjusting the flow so as to minimize as far as possible the effects of the diversion.

It must be kept in mind, however, that a diversion of any considerable amount of water from a drainage basin may constitute a degree of interference with natural flow which may be difficult to readjust.

To be remedial, compensating water must in general consist of water not usable at the time it would naturally pass downstream, often called flood water, which by storage may be used advantageously later, in the lower stages of river.

The extent to which flood water may be withheld temporarily in storage depends upon the kind of use and the condition of the river. Thus a riparian owner using water for power can use this up to his wheel capacity. Ordinarily this wheel capacity may be capable of using up to the flow available 20 to 30 per cent of the time, so that there is flood water available for storage without detriment to the power output. If however the river is already partly developed for storage, all or nearly all the flood waters may be in use and unavailable for compensating storage.

The speaker has called attention to the factor of time of transit. This appears to be quite as important in the matter of compensating flow as the amount of water to be released. The U. S. Supreme Court recognized this in its findings in the Delaware River Case when it required as index points of compensating flow Port Jervis and Trenton, rather than the reservoir sites in the headwaters. The information which the State of New Jersey contributed upon this important matter in the Delaware River Case should be paralleled and confirmed by similar work upon other rivers. As the speaker has intimated, the whole subject of time of transit, including flood waves, steady flow and the various conditions arising from different methods of release of water from reservoirs is a subject which should be given exhaustive study. This may be in part accomplished by laboratory model tests and computations, but actual field tests—which may be possible upon some rivers by cooperating with the power companies—are an important essential.

L. K. SHERMAN:[10] The recent decisions of the U. S. Supreme Court regarding diversions of water between drainage basins on inter-state

[10] Consulting Engineer, Chicago, Ill.

streams establishes a new factor in the allocation of water rights;—the principle of "equitable apportionment."

The application of this principle thus far is dependent upon local facts and situations. It has not been crystallized into a rule of law. It is dependent upon engineering evidence. No apology to the legal profession from engineers discussing this subject is necessary. Mr. Horton's paper is timely and constructive.

Where does the principle of "equitable apportionment" lead? How does it affect and how may it be coordinated with the State laws concerning water rights?

The common law adopted by the older States regards with abhorrence any diversion from one water shed to another. This law considers flowage primarily as an appurtenant of the land and as private property. The common law is frequently against the best public policy and the best utilization of natural resources. It has, however, the merit of being a rule of universal application and it is consistent for all situations.

In many of the Western States the rule of priority of use prevails. A permit from the State is required and hence primarily the flowage is the property of the State. It may grant the individual the right to divert water permanently from a drainage basin.

The rule of "equitable apportionment" of flow of inter-state waters has been an evolutionary process brought about by increased population and increased competition for water. In the drainage canal case Chicago offered to construct compensating works which would fully maintain all alleged navigation and riparian requirements. The principle of "equitable apportionment" had not then been developed. The common law was still sacred and diversion from a water shed, with or without damage and with or without compensation was abhorrent. The transition, through the Connecticut-Massachusetts case and the New Jersey-New York case had not yet taken place. Chicago, like the old martyrs to the Holy Inquisition, was just out of luck because it was on the ground too early.

The rule of "equitable apportionment" has come, at least as between States. It recognizes the equity and necessity for diversion from one water shed to another. It recognizes the necessity for storage and regulation of flow. It is incompatible with the old rule of common law. If the flow as in a state of nature is no longer to be maintained, as between States, then private rights to predetermined flowage in any State does not exist, —at least until a Supreme

Court decision takes place. If diversion of flowage is equitable and proper public policy as between States then it is likewise good public policy within a State. This means that flowage of a stream within a State should be the property of the State and not the property of the riparian owner. The subborn fact remains that the right to flowage under the common law of the State is private property and cannot be diverted by the State without compensation. The value of a particular flowage depends to a large extent upon the limitations which various States have imposed upon any riparian owners common law rights. Such limitations of use include permits, before a dam is built or an obstruction placed, a certificate of convenience and necessity for a public utility or a navigation requirement. These State or Federal limitations may reduce the amount of collectable damages in certain cases, but the fact remains that the common law is a serious barrier to the application of the principle of "equitable apportionment" or diversions between drainage basins.

I have attempted to find what might be some general basis for "equitable apportionment." The following bases, as between States, have been considered:

1. Flowage apportioned according to the relative drainage areas in each State.

2. Apportionment of flowage above a fixed minimum in proportion to the relative drainage areas in each State.

3. Apportionment of flowage above a fixed minimum in proportion to the relative populations in each State.

4. The right of any up-stream State to take any desired flowage above a fixed minimum, but subject to payment of damages to the lower States.

None of these bases are satisfactory. Until engineers and lawyers evolve such a basis, the war for water will go on.

JACOB A. HARMAN:[11] The author is to be commended for the comprehensive review with case references, constructive suggestions and methods of investigating the physical and economical effects and relations of the uses of stream flow.

Public water supplies

In the eastern and mid-western states, it is only in recent years that much serious thought has been given to the economic value and

[11] Consulting Engineer, Peoria, Ill.

conservation of the surplus waters which flow idly or destructively to the sea; but in the west where rainfall is scant a system of appropriation and allotment of water for private and public uses has supplanted the common law rule of "riparian rights" to the use of water in lake and stream. In short, it is now recognized that the available water supply of any region fixes the ultimate limits not only of the agricultural, commercial and industrial developments but likewise of the population that may be sustained.

Many large centers of population have at great cost found it economically necessary to procure their water supplies by diversion from less populous water sheds. The areas from which water is diverted and to which it naturally and legally belongs are thereby limited in potential ultimate development.

Consequently the common law right of the riparian owner "to have the water passing over or along his property in its natural regimen undiminished in flow and unimpaired in quality" has been developed and jealously guarded by courts and legislatures. However, private right must yield to public need and since each State is a sovereign power subject only to the authority vested in the Federal Government by the constitution, it may control and determine the best use of the waters within its borders, but must do so under the common law principle, that such use and control must recognize the right of the down-stream riparian owner and state to the natural flow or stage and/or to an equitable division of such waters. Furthermore, the riparian owner, in exercising his right to use the water, must do so in such a manner as to produce only a minimum or substantially negligible damage or disadvantage to the owner below.

The Delaware River Case

The doctrine of the "equality of rights" of each state under the Federal Constitution and of "equitable division" of the flow of a stream was applied in the recently decided Delaware River case, State of New Jersey vs. State of New York and City of New York as discussed and interpreted in the Report of the Special Master and the opinion of the U. S. Supreme Court. As the author points out, the court followed very closely the common law principle and enjoined the State of New York and the City of New York from diverting any water from the Delaware River watershed in the State of New York, except on the following conditions:—

1. That compensation flow be provided from reservoirs, con-

structed and maintained by the divertor, to the extent of 0.66 c.f.s. per square mile of diversion area when the flow of the Delaware River falls below 0.50 c.f.s. per square mile at Port Jervis New York or at Trenton, New Jersey or at both places.

2. A further compensation, requiring the purification of sewage and industrial wastes at Port Jervis before any diversion may be made.

Under these provisions for protecting the private rights of riparian owners and the public interest of the States of New Jersey and Pennsylvania, New York City may divert from the Delaware watershed not to exceed the equivalent of 440 m.g.d., but since the United States is not a party to the suit and not bound by any decree, this taking is "necessarily subject to the paramount authority of Congress over navigation and navigable waters." Furthermore, the U. S. Supreme Court retains jurisdiction to enforce or modify any of the provisions of the decree.

The explicit and implied obligations and the uncertain tenure jeopardize the investment of the divertor and would naturally induce the development of a water supply from areas under complete jurisdiction of the State of New York, even at considerably greater cost per unit of service.

Economic uses of streams

The author's comprehensive list of potential economic uses applies in part and in variable degrees to even the smaller tributaries, and the relative importance of each use may become a determining factor in the development.

In the eastern and central western states, the following uses absorb the public and private interests.

1. Navigation,
2. Public and industrial water supply,
3. Sanitation,
4. Water power,
5. Fisheries,
6. Recreation and
7. Flood control

The development of the western plains, mountains and coast country, has depended upon the appropriation of the stream flow for agriculture, water power (mining and industrial) and public water supplies.

The earlier developments appropriated and depended upon the ordinary or low water stream flow, but later developments conserve the flood water impounded behind dams and feed out for power, irrigation or public water supply as needed. Many communities are sustained upon water from other watersheds. Los Angeles, probably the most conspicuous example, with its present water supply taken in part from trans-mountain streams more than 250 miles away, is preparing to take additional needed supply from a more distant but available source provided by the Hoover Dam on the Colorado River.

Local developments with concentrations of population and particularly the transportation facilities determine and frequently greatly modify the relative importance of the numerous economic uses to which the water of any stream may be applied.

The provision for future and continuing consideration by the court of the problems in the Delaware River case and the Chicago Diversion case may therefore keep the way open for best use of the waters involved by maintaining a balanced economic development of the regions served. Irrevocable diversions authorized in the light of the knowledge and judgment of today may lead to irrepairable injury to the future best economic use of the water. Planning for conservation of water, therefore, in addition to consideration of the legal water rights, should proceed upon a flexible theory of adaptation and conversion from one major use to another as economic conditions may change.

The writer concurs with the author in that the "paramount" use may not be domestic public water supplies, for people in cities must live by commerce and industry and can have only local markets without moderate cost transportation. Following the use of water a little farther, the cheapest form of transportation known is navigation and all of the greatest commercial and industrial communities are on navigable waters.

Effects of diversion and inversion

The diversion or inversion of either variable or uniform flows produces changes in the regimen of any stream that may profoundly modify its value for one or more of the normal uses. The changes may be either all benefits or all damages, but usually require consideration of benefits, damages and costs to determine the economic balance. The problem presented is involved and difficult. Methods of solution of hydraulic effects have not been developed to a satis-

factory understanding and many unproven theories persist to vex the investigator. One difficulty usually is inadequate hydrological data and survey data.

The contour or available storage of any lake or stream subject to variable flow produces a most profound effect on the stage and outflow. It is only when inflows from all sources are sustained at uniform rates for sufficient time to establish equal outflow that the usual methods of hydraulic computations of stream flow apply.

The velocity of flow at normal stages, duration and rate of increased inflow, rate of flood wave travel, length and surface area of stream, lake, reservoirs or pools and the outflow rate all contribute and merge into and profoundly modify the stage or stream height which in turn is a function of storage and outflow.

A useful step in the solution of the effect on stage of variable inflows has been expressed in a formula[12] developed by the writer based upon the accepted fundamental formula,

$$\text{Inflow} = \text{Storage} + \text{Outflow}$$

and is as follows:

$$h = \frac{I - Q_1}{A + \frac{1}{2} q}$$

in which

h = the change in stage in feet for the unit of time adopted.

I = inflow in acre feet days or equivalent for the unit of time adopted.

Q_1 = the outflow at the stage at the beginning of the day in acre feet days or the equivalent for the unit of time adopted.

A = the average storage area in acres at the stages and for the unit of time.

q = the change in outflow for one foot change in depth at the stage at beginning of day, in acre feet days or equivalent for the unit of time adopted.

The formula as written is intended to be used with unit of time one day, and inflow and outflow converted from c.f.s. days to acre-foot days. Wherever daily gauge readings, reliable discharge rating curves and storage curves are available, this formula may be used:

1. To compute the storage values for the given reach of the river above the gauging station for any change in stage.

[12] This formula is published in *Flood Control Report*, Illinois Division of Waterways, June, 1929, page 111.

2. Having the storage values, a hydrograph may be computed with modified inflows.
3. The difference between the observed stage and the computed stage for any day will represent the effect of increase or decrease of inflow at the gauging station.

This method may be applied in the development of hydrographs for modified flows. In computing storage, a sheet of water of the depth "h" and the areas "A" represents the storage effect manifest for that unit of time at the gauging station. Any flood wave approaching the gauging station, a part of the inflow at some point above, is manifest on the gauge through successive units of time and produces an increase of stage, storage and outflow. By choosing such a distance up-stream from the gauging station that the changes in surface slope are small or negligible for the unit of time used in the computation, we use algebraically the above differential equation with the resulting computation as accurate as the original data permits. With variable and irregular inflows as found in nature, the summation of the direct computations of the unit-of-time effects furnishes a practical method of determining changes in stages and discharges due to variable inflow and constant or variable diversion or inversion flows.

The author's suggestion for comprehensive engineering and economic studies to determine the most valuable uses of the stream flow and diversion effects is not of mere academic interest, but of the greatest practical value.

EXPANSION OF THE LOS ANGELES DISTRIBUTION SYSTEM

By W. W. Hurlbut[1]

The municipal water system of the City of Los Angeles dates back to February, 1902, at which time the City, through purchase, acquired the private water companies serving the city with water. The consideration of acquiring these works was $2,000,000. At that date the population of Los Angeles was 130,000 and the area 43 square miles.

For the past thirty years the city of Los Angeles has had an accelerated rate of growth and there is no indication at the present time that such rate is to be retarded materially. The population of Los Angeles according to the 1930 census was 1,238,000 people, and its present area is 450.5 square miles. The distribution system as of today comprises 3,700 miles of pipe lines greater than 4 inches in diameter, and the value of the works as of today is $120,000,000.

By referring to figures 1 and 2 the elements of the water system are plainly set forth, showing the growth of the system from 1900 to date. The problem of attaining sufficient water and properly distributing it through a distribution system capable of extending in any direction has been and still is an active one.

The sources of supply of the City of Los Angeles are the Los Angeles River, the Los Angeles Owens River Aqueduct, and the local wells.

Up to the year 1910 the City Water Works system's growth was definitely limited by the quantity of water available from the Los Angeles River and City well supply,—and the distributing system therefore centered on the Los Angeles River as the chief source of supply and radiated out from this source.

In 1905 a well supply was developed at Slauson and Compton Avenues and water pumped directly into the mains. Another well supply and pumps were installed at Figueroa Street and Slauson Avenue in 1907 to aid the Compton Avenue plant to supply the summer demand in this southern section of the city. These two

[1] Engineer of Water Works, Department of Water and Power, Los Angeles, Calif.

ground water plants served for the system *first* as a new source of supply, and *second* as a way of boosting the pressures in that area north of Slauson Avenue. They served the same purpose as a balancing reservoir or kickback tank does to any distributing system, but they had the added advantage that no costly inlet line was

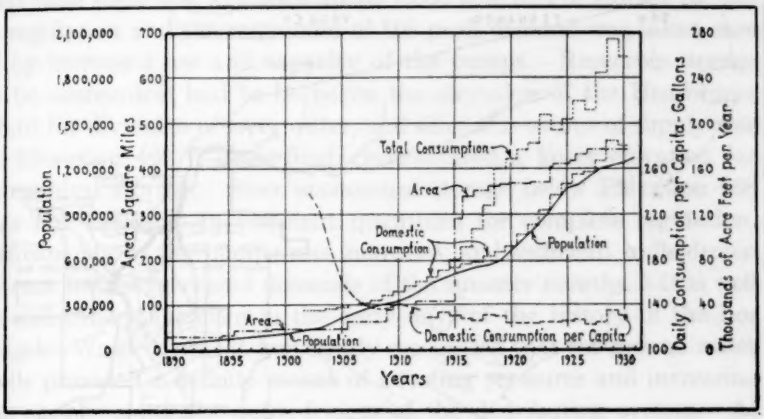

FIG. 1. POPULATION, AREA, DOMESTIC CONSUMPTION, TOTAL CONSUMPTION AND PER CAPITA CONSUMPTION

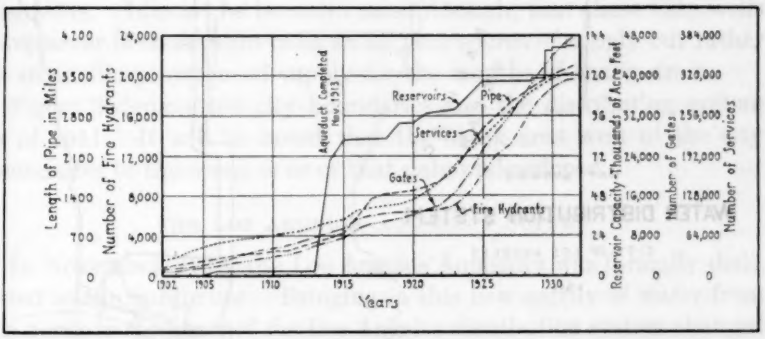

FIG. 2. NUMBER OF SERVICES, FIRE HYDRANTS, GATES, LENGTH OF PIPE IN MILES, AND RESERVOIR CAPACITIES IN ACRE FEET

required to adapt them for use in the system. It is on record that the Slauson Avenue plant paid for itself from water revenue during the first year following its installation.

These wells and pumping plants served admirably to bolster up the service and pressures in this outer end of the distribution system

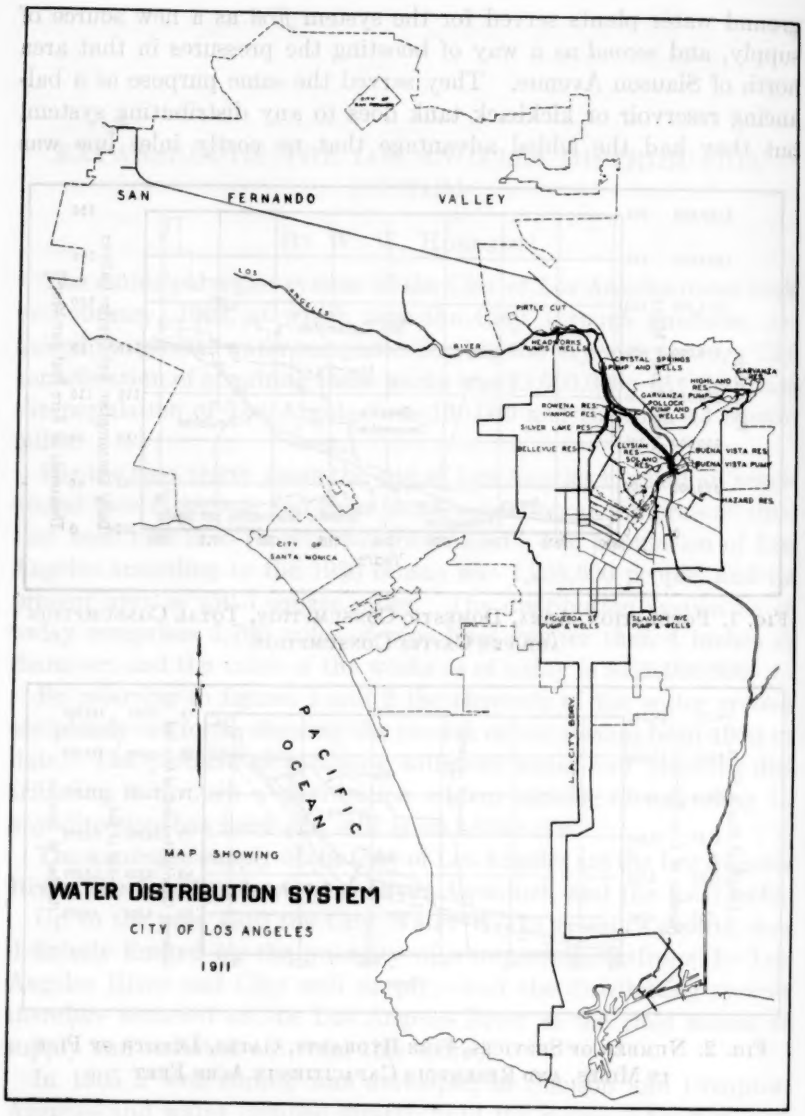

FIG. 3

but they had one serious drawback. Because of the steady falling of the ground water level in the area in which they were located they could not be considered as a permanent source of supply.

The mean consumptive use of water in 1910 was 70 cubic feet per second, and according to today's standards there should be about 5,600 acre feet of storage available for the efficient regulation of such a consumption of water. Our experience shows that 80 acre feet of storage are required to regulate one second foot of consumption. At that time there was available about 1,500 acre feet of storage for use in regulation and the remainder of the peak demand was taken care of by increased use and capacity of the pumps. Reservoir storage to be economical had to be below the elevation of the Headworks Plant for diversion of river water, and since this source of supply was at Elevation 468 it accordingly necessitated a lower elevation for economical storage. Since economical storage below Elevation 468 was not available in sufficient quantities for complete regulation, the capacity of the pumps was increased, and sufficient wells driven to care for the increased demands of the summer months. It is well to remember that even in the early days of the history of the Los Angeles Water Works it was clearly recognized that the ground water wells provided a definite means of boosting pressures and increasing the supply along the outer fringes of the distributing system. As has been stated previously, these wells performed the duty of kick-back tanks or balancing reservoirs at a fraction of the cost of such reservoirs. It must be borne in mind, though, that these City wells have never been thought of as an all year source of supply but rather as an auxiliary system of supply for the months of heavy draft.

Figure 3 depicts the city boundaries and the distributing system as of 1911. It will be noted that the blank area west of the city boundaries to the ocean is as of that date undeveloped.

THE LOS ANGELES AQUEDUCT SUPPLY

In November, 1913, the Los Angeles Aqueduct was formally dedicated to the public use. Bringing in this new supply of water from the north to the heart of the Los Angeles distribution system changed somewhat the manner of planning for the expansion of the distribution system as a whole. This water enters the city of Los Angeles at an elevation of 1,460 feet above mean sea level. The elevation of the Los Angeles River at the highest point of diversion is Elevation 468. That is to say, the new aqueduct provided water at the north boundary of the city at an elevation approximately 1,000 feet higher than that provided for by gravity by the original system of water works. The capacity of this new aqueduct was 400 second feet which

represented an addition to the water supply of five and one-half times the original river supply. The Los Angeles River supplied its own storage in its gravels to take care of the many demands of the summer months, but the new Los Angeles Aqueduct needed storage at the city end of the aqueduct to make it an efficient and safe source of supply during all seasons. Consequently, reservoir sites were acquired and reservoirs constructed to provide storage at the city end of the aqueduct. These reservoirs were built on the best possible sites available and the location of these reservoirs is a prime factor in the design of the Los Angeles Distribution System. These reservoirs and their outlets are the focal points of the distribution system. In the interests of economy the Los Angeles Aqueduct was built as a single channel, or as a single supply conduit. But the Los Angeles Distribution System is not designed to place too great reliance on any particular trunk line, but is designed as a system such that its various units will function under normal or emergency conditions that may be placed upon it.

Shortly after entering the city's boundaries the waters of the Los Angeles Aqueduct are decentralized. The decentralization serves several purposes. First, the waters are conducted to the various storage reservoirs in the San Fernando Valley and in the Santa Monica Mountains. These reservoirs have a combined capacity of 52,000 acre feet of water and they provide the necessary close-in storage required for the city of Los Angeles. The lines inter-connecting these reservoirs make possible the filling of the lower reservoirs and in addition provide for the demands for irrigation in the San Fernando Valley. They therefore make possible the large revenue obtained from the sale of water for irrigation and at the same time add to the flexibility and safety of the entire system.

The distribution system of the San Fernando Valley was primarily designed for purposes of irrigation, and secondarily to provide water for domestic use to the various small communities within its borders.

Domestic use alone would not justify the system of large trunk lines which extend across the valley from east to west and from north to south. However, the use for irrigation does justify their construction. The annual report on financial condition shows sufficient water revenue derived from irrigation to amply justify the system of irrigation lines if they served no other purpose. But they do serve several different purposes other than for irrigation. They provide for the conveyance of water to the reservoirs from which the Main City Area

is directly served; they provide for domestic service to the smaller communities in the San Fernando Valley; and indirectly through their use for irrigation they provide for replenishing the great underground reservoir which underlies the San Fernando Valley. It is estimated that at least 35 percent of the waters used for irrigation percolate into the underground gravels of the valley where they build up the storage in that great natural underground storage reservoir. This water finds its way into the Los Angeles River from where it is taken for use in the distribution system as needed.

REPLENISHING UNDERGROUND GRAVELS

During the winter months thousands of acre feet of water are taken from the City Trunk Line and discharged over large prepared beds of sand and gravel known as "Spreading Grounds." These grounds have a tremendous capacity for the rapid absorption of water into the underground gravels of the valley. The rate of absorption is 2 second feet per acre of spreading area. These spreading grounds were designed to provide for the rapid induction into the underground water basin of the large quantities of water that in some years are available from the Owens Valley supply. The value of this underground storage to the safety of the Los Angeles water works system is almost incalculable. Equivalent reservoir storage, if available, would cost many millions of dollars and then could not possibly deliver the water to the system in such a pure state as it is delivered from the underground storage. To an engineer who had not made a study of this underground basin for years a doubt might naturally arise as to whether all the water induced into the ground could again be recovered, but a careful study of the flow of the Los Angeles River over a period of many years proves that water which sinks into the valley soil can be recovered again either via the Los Angeles River water works system or via pumps and wells in the valley itself. The Los Angeles River is essentially an underground stream which is fed by the gravels of the San Fernando Valley, and comes to the surface at the lower bottlenecked end of the valley.

THE DISTRIBUTION SYSTEM AND RESERVOIRS

The distribution system for the Main City Area of the city of Los Angeles originates in the Los Angeles River and the four large reservoirs situated in the ring formed by the Santa Monica Mountains and the Los Feliz Hills. These major points of distribution extend

along a line about fourteen miles long and from them come the large trunk lines forming the skeleton of the distribution system for the city. From west to east these reservoirs are as follows:

RESERVOIR	ELEVATION	CAPACITY (ACRE FEET)
Stone Canyon	841	8,132
Franklin	573	1,052
Hollywood	715	4,240
Silver Lake	447	2,162
Rowena	448.5	94
Elysian	438	32
Buena Vista	374	39

From these reservoirs the greater part of the city is supplied by gravity water, and from them also the water is pumped to the high service areas. These reservoirs are all filled by gravity lines. Those areas, extending roughly from Elevation 80 to Elevation 750 are supplied by gravity from these reservoirs. Those areas extending from Elevation 600 to Elevation 1,400 are supplied from waters pumped from these reservoirs or their feeders.

The larger reservoirs in the above list were primarily constructed to provide part of the necessary storage for the Los Angeles Aqueduct. They are considered as insurance against failure of the aqueduct or conduit leading from a more or less remote source of supply.

First, these reservoirs are storage reservoirs, and they have been designed and constructed with this end in view. Secondly, however, they become distribution reservoirs and their outlet lines become focal points of the distribution system. Their use as such very effectively decentralizes the trunk lines supplying the smaller mains of the distribution system and thereby makes the supply at all times as certain and safe as is possible. These reservoirs provided much needed storage for safety and at the same time permitted the construction of smaller major supply trunk lines extending from the San Fernando reservoirs to the Santa Monica Mountains reservoirs than would have been required without them.

We wish to bring out the point here that each city has its own peculiar problems of water works and water distribution different from those of any other city of equal population. The engineers charged with the design and operation of the larger water systems of the country are becoming agreed on some basic points of design.

They believe that the minimum requirements of serving a residential area should be a system of 6- and 8-inch mains, with 16- and 12-inch mains interlaced at regular intervals in the grid system. In this they are in agreement with the Board of Fire Underwriters, who are in a large measure responsible for the increase of efficiency and safety in the modern American water works system. They are agreed that elevated tanks, or reservoirs at points of vantage, present opportunities of better service at less expense than a line without such a balancing reservoir at or near its end. They agree that pressures of 35 to 60 pounds in the distributing mains are desirable for ordinary use, but that where pressure equipment is used by the fire department the pressure in the mains may be pulled lower to 10 pounds to provide a very heavy draft; and they no doubt agree that the topography of the area or city to be served is decidedly the one chief factor which governs the design. And in such an arid section of this country as Southern California, even such things as the decisions of the courts on disputes concerning the ownership of water rights will have their effect on the design of the distribution system.

The basic fact on which a distributing system for a water works is designed is the same as on which all major engineering works are designed,—that is, absolute faith that they will economically serve a useful purpose and a real need. Here in Southern California, where water is more precious than in other parts of America, is the underlying conviction, a faith if you wish to call it so, that any water which is brought into this land will attract people to its use in sufficient numbers to make such projects economically sound. It has been the past experience in the city of Los Angeles that the population has a way of settling between the trunk lines in a comparatively short time after they have been constructed.

The previous paragraphs have been in the consideration of the distribution system in the broader sense of providing water to the area 450 square miles in extent, which is the city of Los Angeles. In another sense this great distribution system is in reality made up of several smaller distribution systems, each of which presents its own problems of design.

LOCAL AREA DISTRIBUTION

Let us assume a problem for the supply of a certain definite area of the city with a distribution system.

In cities with greater than 200,000 population the Underwriters'

requirements are for 12,000 gallons per minute, with 2,000 to 8,000 g.p.m. available for a second fire,—that is, in cities of upwards of 200,000 to 1,000,000 and more inhabitants, the charge for fire protection (so far as providing for it in the trunk lines of the distributing system is concerned) becomes almost negligible. However, do not forget that this statement applies to the larger mains, say 24-inch and larger, and is decidedly not true when the vast grid system of 6-, 8-, 12- and 16-inch mains is considered.

Consider, for example, all that area of Los Angeles extending, roughly, south of the Santa Monica Mountains south of Silver Lake Reservoir and west of the Los Angeles River, and extending south as far as 120th Street, and to the west to the boundary of the city of Santa Monica. This area contains about 58,000 acres or 90 square miles of land served by the City of Los Angeles Water Department's Main City Distribution System. The term "Main City" is here used to distinguish this inter-connected system of mains and reservoirs from the systems of Venice, the Harbor District, and the San Fernando Valley. This area has a population of approximately 1,000,000 people. Consider the trunk lines supplying this area with its 1,000,000 inhabitants. The table below lists the mains feeding into this area:

TRUNK LINE	SIZE	EQUIVALENT IN TERMS OF 36 INCH MAIN
	inches	
Stone Canyon Trunk Line	30	0.62
Stone Canyon Reservoir—Pico Trunk Line	36	1.00
Franklin Canyon Reservoir Outlet	36	1.00
	40	1.32
Hollywood Reservoir Outlet	40	1.32
Rowena Reservoir Outlet	42	1.50
Bellevue Reservoir Outlet	30	0.62
Buena Vista Reservoir Outlet	40	1.32
Silver Lake Reservoir Outlet	36	1.00
Western Avenue Tank Outlet	36	1.00
		10.70

These are the mains that supply the peak draft to this area. They are equivalent to ten mains 36 inches in diameter or one single main 114 inches in diameter. If we consider the trunk line capacity required for a city of 1,000,000 people, at the rate of 130 gallons per

capita per day, with a peak demand of 2.5 times the mean yearly rate of use, and with an assumed economic velocity of 7 feet per second we shall find that the mean rate of use for the total area will be 200 second feet, the maximum domestic peak use 500 second feet, and the equivalent single main required equal to one 114 inches in diameter.

An examination of the records of use on July 3, 1931, showed a peak use in this area of about 364 second feet. The mean yearly use for this area has been determined to be about 200 second feet, more or less, which would be at the rate of 130 gallons per capita per day. For the entire area, then, the ratio of peak demand to mean yearly use is 192 percent, which is considered about correct for this large area. The ratio of 2.5 to 1, when considered for a very large area, is too liberal a factor to be used, but in the case of design for small parts of the distribution system is essentially correct.

The list of trunk lines previously given in this discussion shows the extent to which the distribution system for the Main City Area has been decentralized. Instead of supplying the system from one large trunk line it is supplied from ten or more smaller lines, each one backed up by sufficient storage to act for several days as direct sources of supply to the system,—and so interconnected that the failure of any one of these lines shall not paralyze any great portion of the distribution system.

Assuming that this assumption of 7 feet per second velocity to be approximately a correct one, it would follow from the preceding that this Main City Area is adequately provided with *major* trunk lines to distribute a mean yearly use of 200 second feet and a peak use of 500 second feet of water. On the basis of a mean yearly use of 200 second feet and a peak use of 364 second feet, the existing mains would have capacity enough for a mean yearly use of 260 second feet. If the use of water increased in the future as it has in the past the existing major trunk lines will be taxed to capacity in the year 1936,—that is, within four years from the present time.

It is not intended to convey that this so-called economic velocity of 7 feet per second is correct in this case or any other case, but in a pumping line designed for continuous pumping it is well to recognize that pipe of the most economical diameter can readily be computed and so also the most economical velocity. These economic velocities are found within rather narrow limits, say of from 2 feet per second in the smaller pipes to 5 feet per second in the larger pipes. Just as

there are economic velocities in pumping mains there are economic velocities in gravity mains, and in a large system this fact become apparent more and more as the system is expanded. The economic velocities in a gravity system are well recognized, particularly in the cases where the development along the line of hydroelectric power is possible. To put it in another way, falling water has a definite economic value, and every foot of head is worth something.

GROWTH OF SYSTEM

At this point reference is made to figure 4, which shows the city of Los Angeles as of today, with the great area lying west of the old city boundary as referred to in figure 3 having been largely filled in by supply mains and the distribution system. It will be noted that this map gives a very vivid picture of the Los Angeles City Distribution System, extending in elevation from sea level to 1,450 feet above sea level, and comprising twenty-four zones of supply,—nine gravity and fifteen pumped, which are served by twenty-seven reservoirs, thirty-seven tanks, and forty pumping plants, which double ends the supply of the entire grid system.

It is anticipated that the city of Los Angeles within the coming ten or fifteen years will have several millions of people within its present boundaries and be built up quite solidly from the downtown business district to the sea. But as might be expected, some of these areas will experience a much more rapid growth than others. The water distribution service for these areas can be supplied from the line of reservoirs extending fourteen miles in length along the line of the Santa Monica Mountains and the Los Feliz Hills as the need arises. These several focal points of distribution permit the trunk lines to adjoining areas to complement one another, and in the certain event of necessary future replacement permit a comparatively simple solution of serving the district supplied by the main to be replaced.

The water works engineers of the Los Angeles City System anticipate the entry of Colorado River water into the City system at approximately the year 1940 and this provision has been made possible through the cooperation of the City of Los Angeles and the Metropolitan Water District of Southern California, which have definitely voted bonds to finance the Los Angeles-Colorado River Metropolitan Aqueduct.

The existing Los Angeles Aqueduct enters the city from the north, whereas the trunk lines from the Colorado River storage reservoirs

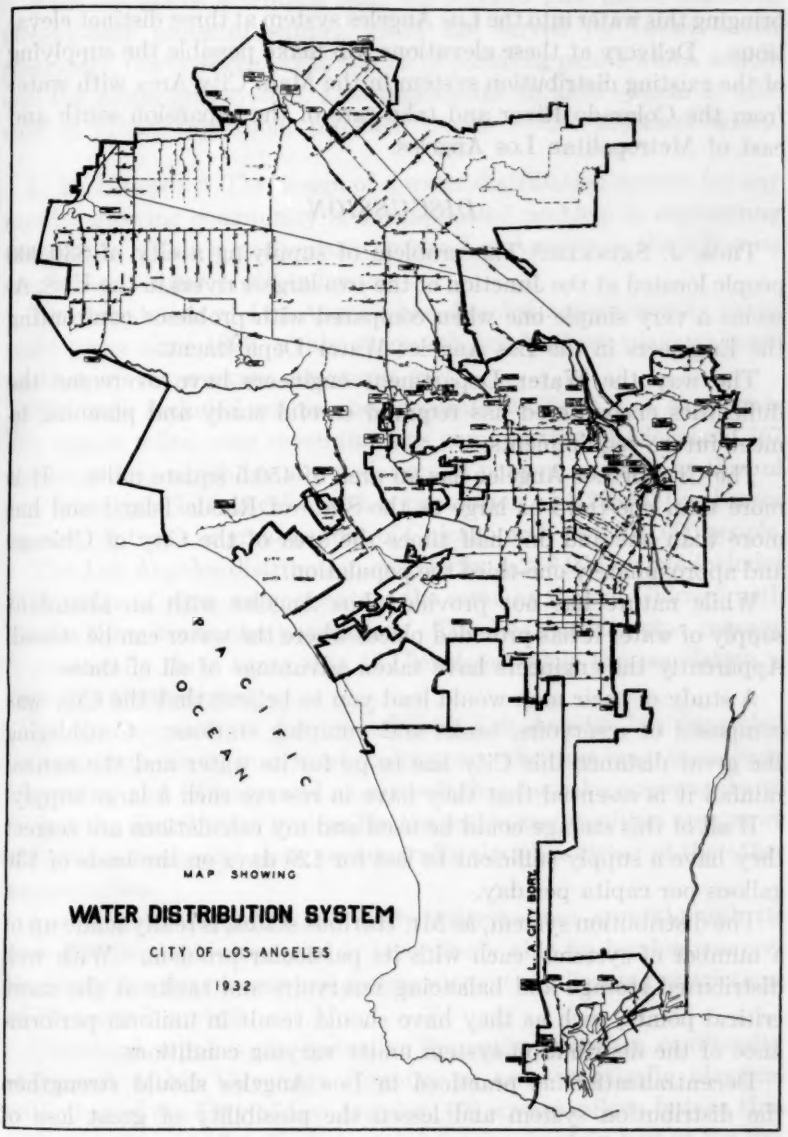

FIG. 4

will enter the city at points along its eastern boundary. Tentative plans, which are still decidedly in the preliminary stages, anticipate

bringing this water into the Los Angeles system at three distinct elevations. Delivery at these elevations will make possible the supplying of the existing distribution system in the Main City Area with water from the Colorado River and take care of the expansion south and east of Metropolitan Los Angeles.

DISCUSSION

THOS. J. SKINKER:[2] The problem of supplying a city of 830,000 people located at the Junction of the two largest rivers in the U. S. A. seems a very simple one when compared with problems confronting the Engineers in the Los Angeles Water Department.

The way the Water Department engineers have overcome the difficulties encountered has required careful study and planning to meet future requirements.

The City of Los Angeles has an area of 450.5 square miles. It is more than one-third as large as the State of Rhode Island and has more than one and one-half times the area of the City of Chicago and approximately one-third the population.

While nature has not provided Los Angeles with an abundant supply of water it has provided places where the water can be stored. Apparently the engineers have taken advantage of all of these.

A study of their map would lead you to believe that the City was composed of reservoirs, tanks and pumping stations. Considering the great distance this City has to go for its water and the annual rainfall it is essential that they have in reserve such a large supply.

If all of this storage could be used and my calculations are correct they have a supply sufficient to last for 120 days on the basis of 130 gallons per capita per day.

The distribution system, as Mr. Hurlbut states, is really made up of a number of systems, each with its particular problem. With well distributed storage and balancing reservoirs and tanks at the more critical points, such as they have should result in uniform performance of the distribution system under varying conditions.

Decentralization as practiced in Los Angeles should strengthen the distribution system and lessen the possibility of great loss of water.

It would be interesting to know how the twenty-four zones are separated. Is this done by closed valves or do they have pressure regulators between zones?

[2] Engineer in Charge of Distribution, St. Louis, Mo.

With elevations extending from sea level to 1450 above sea level, twenty-four zones of supply which are served by twenty-seven reservoirs, thirty-seven tanks and forty pumping plants which double end the entire grid system, Los Angeles has an ideal system to take care of future expansion, provided it can secure sufficient water.

L. R. HOWSON:[3] The design of a water distribution system for any rapidly growing community is a complicated problem in engineering economics. This is particularly true in Los Angeles as the following comparative figures will indicate.

The population of Los Angeles has doubled approximately every eight years since 1890, a rate of growth nearly three times as rapid as that of the average American city.

The area included within the corporate limits of Los Angeles is 450 square miles, over seven times as great as that of Cleveland (67 square miles) which has a population 75 percent as great as that of Los Angeles. Los Angeles with a population of 1,238,000 embraces an area $2\frac{1}{4}$ times as great as that of Chicago with its 3,376,000 people.

The Los Angeles distribution system for 1,238,000 people includes 3700 miles of pipe, approximately the same as that of Chicago with nearly three times the population. The Los Angeles pipe mileage is practically three times that of Cleveland, while the population is only one-third larger.

As the comparative figures indicate, Los Angeles has expanded its corporate limits in a much more comprehensive way than have Cleveland and Chicago and as a result, has had the opportunity to design the distribution system for the entire metropolitan area more comprehensively and more economically than has either of the other communities.

Los Angeles furnishes the entire metropolitan area through its own distribution system and is accordingly able to develop comprehensively for the present and future without the limiting restrictions of diversified control.

Cleveland supplies the water for the entire suburban community embraced within Cuyahoga County, an area practically identical with that of the City of Los Angeles (450 square miles), but, in that area, the City of Cleveland owns only approximately half of the distribution mains, the balance being owned by the separate municipalities and sixteen Sanitary Districts.

[3] Alvord, Burdick and Howson, Consulting Engineers, La Grange, Ill.

In the Chicago Metropolitan District, the Chicago mains terminate at the corporate limits. Some 30 or more communities with populations varying from a few hundred to as much as 100,000 people have individual systems designed largely as integrated units and without respect to coordination with the City of Chicago.

Of these three systems, Los Angeles alone has opportunity for an unrestricted, economically developed, comprehensive plan of distribution for the entire metropolitan area.

The utilization of the underground gravels for storage of surplus water during the winter months and the abstraction of that water when needed direct from the gravel or from the Los Angeles River to which the surplus percolates is of great economic advantage. Few cities have this opportunity. A somewhat similar situation exists at Des Moines, Ia. where the supply is secured from infiltration galleries located adjacent to and paralleling the river. The gallery system is 14,000 feet in length, largely 4 feet in diameter and located at a depth of some 10 to 15 feet below the river bed. The area of the river bed paralleling the galleries is some 55 acres. At certain seasons of the year, due to the silting of the river bed, the draft which the galleries secure from the river decreases to as small an amount as 200,000 gallons per acre per day. At such times, it is the practice to lift water from the river and discharge it over specially prepared flooding basins having an area of 44 acres preceded by 9 acres of sedimentation area. The gallery lies between this flooding area and the river. The flooding pumping installation has a capacity of 30 million gallons per day. The system has demonstrated its ability to supply approximately one million gallons per day per acre of flooding area. In extreme years, nearly 50 percent of the entire amount of water used at Des Moines has been secured by this means. The capacity of the Des Moines "spreading grounds" is apparently somewhat less than that at Los Angeles possibly due in part to the silt carried in the river water at Des Moines. This is periodically removed from the spreading grounds by scrapers.

The availability of reservoir sites has made practicable at Los Angeles an effective decentralization of the trunk line system which undoubtedly has materially reduced the expenditures which would otherwise have been required.

The Los Angeles distribution system is an interesting combination of gravel storage, feeder mains and advantageously located distributing reservoirs in a community of unusually rapid growth. Without

this combination, the cost of distribution for equal service would have been much greater.

SAMUEL B. MORRIS:[4] The major pressure districts in the city are separated by closed valves. There are in the hill sections, a number of comparatively small pressure service areas in separated zones and these are separated by regulating valves, but that is in a comparatively small portion of the system. Most of the pressure districts are separated by closed gate valves.

I would like to make one additional observation. Mr. Howson's statement is partially true regarding the fact that Los Angeles controls so much territory and is able to plan its distribution system for the future. Los Angeles represents more than half, I should say sixty per cent, of our population there, but we have something like forty per cent comprising about twenty-five other cities immediately adjoining Los Angeles and making one metropolitan area of something like two and a quarter million. Pasadena is one of these smaller communities. Long Beach with a population of one hundred and fifty thousand is the largest outside of Los Angeles and each of these other communities has its own source of water supply and has its own distribution system.

We have recently combined into a metropolitan water district to bring water from the Colorado River. That purpose, however, is only to procure the supply of water and the distribution of it is still planned to be carried out by each separate community.

[4] Chief Engineer, Water Department, Pasadena, Calif.

CAUSES OF WASTE IN DISTRIBUTION SYSTEMS

By Alfred E. Skinner[1]

The former prevalent belief that the supply of water was inexhaustible naturally produced carelessness in construction, control and use. Improvements in construction, meters and use have progressed considerably, but we all know that conditions are not ideal, except in comparatively few systems, and that losses of water still continue.

Water losses (by which is meant all losses in supplying water to the consumers, whether caused by leaks, loss of revenue or unnecessary expenditures) may be divided into the following classes:

Underground leaks of the mains and services
House waste and fixture leakage
Leakage from stand-pipes and reservoirs
Leaking pressure boundary valves
Meter under-registration
Unauthorized use
Pump slip and poorly adapted pumps

It is not possible to arrange these losses in the order of importance, since in one city underground leakage may be greater than house waste, while in another city of similar character, the reverse may be true.

Underground leakage is usually the greatest source of water loss throughout the country. While house waste and fixture leakage are open and above board in their habits, the underground leak pursues its course secretly and seeps its way along the pipe trench to a possible outlet into a sewer or to a crevice in the rock through which the trench was cut.

As pavements improve and heavy concrete bases seal the surface, the water may gradually excavate a hole under the pavement for a heavy truck to crash through, or the leak may work its way outside the road bed and undermine the foundation of a building. Whatever its course, unless the leak comes to the surface or its presence is dis-

[1]Western Manager, The Pitometer Company, Engineers, Chicago, Ill.

covered by accident, year after year the leak continues, in many cases increasing in size, and always wasting possible profits of the Water Department. This insidious loss may continue until the total leakage is so enormous that large sums of money must be expended to develop new sources of supply, construct new feed lines, increase the sizes of the distribution mains or all of these combined.

In every water works system there is an indeterminate amount of loss, sometimes called unavoidable leakage, which is distributed over the system in such small quantities that it is not economical to spend the money necessary to locate and repair the individual leaks because of the small saving to be made. However only a careful investigation can determine if this loss really is unavoidable.

JOINT LEAKS

Joint leaks, the most common type of main leakage found, are of two types: those where the jointing material has been forced out of the joint or has deteriorated, and those where the joint has pulled apart. The first type is caused by careless preparation, pouring or calking. Improper preparation of jointing compounds destroys their qualities. A poor joint may be caused by not having enough lead in the ladle to completely fill the joint or to an obstruction which prevents the hot lead from running properly. A bad joint is sometimes due to poor yarning which allows the jointing material to escape into the pipe, not only requiring an excess of material, but forming an obstruction in the pipe. If such a joint is at a valve, the material may prevent the proper operation of the valve. Calking affects the lead for only a limited distance from the outside end of the bell. Due to its contraction in cooling there is no close bond between the lead and the pipe so that, unless the calking is properly done, the whole ring of lead may start, allowing considerable water to escape. This movement is of course limited by the groove in the bell and the bead on the spigot.

The cause of the second type of joint leak, where the pipe is pulled apart, usually is due to faulty construction. A settlement on either, or both sides, of a joint may open the top or a settlement under the joint may open the bottom. Entire separation of a joint is rare in a straight pipe unless due to a washout or similar accident; but at bends, joints are sometimes pulled apart because of improper bracing. The same thing may happen in short branches and blow-offs not sufficiently braced.

BROKEN MAINS

Another frequent source of leakage is due to broken mains. Such breaks are of several types among which may be mentioned the following:

Transverse break in which the pipe is broken entirely across. This is almost always caused by a hard spot, usually a stone or piece of rock, under the point of breakage with softer foundation on each side allowing uneven settlement.

Crack in the spigot end. This is usually caused by careless handling of the pipe during transportation or laying. Since it is held firmly in the bell, there is not much chance for the split to spread under pressure; but under certain conditions a considerable amount of water can escape from such a crack.

Crack in bell end. This type is more serious and may be caused by careless handling, but too heavy calking may make the crack. Such a crack tends to open under pressure and may run back into the pipe, causing serious leakage from the crack itself or from a hole which may break out along the original crack.

Large piece broken out of a pipe. This may be due to a crack as mentioned above or to some inherent weakness which makes itself known under some especially great stress such a water hammer. In such a case the whole line may be subjected to stresses considerably above the nominal limit of the pipe and if there is a defect or weak point, failure at that point is liable to occur.

Holes in the pipe. These are often found and may be due to sand holes, corrosion, electrolysis or erosion. Sand holes are not easy to find after the pipe is coated, and they are rarely the cause of leakage, since the area of the hole is small and the iron, while thinner at that point than it should be, is still of sufficient strength to hold the pressure. Also it is not usual to find holes in cast iron pipe caused by corrosion, except in cases where the main is laid in very corrosive soil. Holes in mains due to stray electric currents have in the past caused considerable leakage, but electrolysis surveys in recent years have done much to eliminate this source of trouble. In some soils where sharp sand is present, a slight pin hole leak, especially in a joint may cause considerable leakage since the sand and water exercise a severe erosive action if the stream hits the pipe. Such erosions have an appearance similar to the deterioration caused by electrolysis. While it does occur in cast iron mains, it is more often seen in lead services or in brass cocks and goose necks.

Defective pipes. These may allow leaks to occur. The sand holes have been mentioned, but are of small importance. In the early days pipes were perhaps never entirely concentric and one side was much thinner than the other. Many of these pipes have stood up under considerably greater pressure than was expected, but sometimes one breaks due to lack of metal in the weak side.

SERVICE LEAKS

Service leaks are of many varieties. In tapping the main, the drill may be over-run, and the cock, screwed down to the face of the pipe, does not take up tightly enough to hold water. Wiped joint leaks are very common and may be due to settlement, causing a crack, or to poor workmanship. Galvanic action between the metals may in course of time destroy enough metal to cause a leak. Leaks at the curb-cock may be due to any of the above causes or to careless handling of a shut-off key. These leaks are all very apt to increase in size by sand blast action of grit picked up by the water hitting the brass or lead. The causes of leaks in the service pipes themselves are too well known to need more than passing mention. Corrosion of iron pipe, either from the inside or outside, and freezing are perhaps the most usual reasons for failure but water hammer caused by quick closing valves sometimes is to blame. Leaks at the meter are usually due to careless workmanship in making up the joints.

MISCELLANEOUS LEAKS

Underground leakage may also be found in glands of valves when the packing has dried or blown out.

A somewhat similar loss occurs in some types of hydrants in which the valve may be slightly open without closing the drain, thus allowing a considerable amount of water to escape.

Check valves may be a source of loss unless they are carefully examined and cleaned at short intervals. Even then a single-check valve may be open for a considerable loss of water if anything lodges under the flap. In a distribution system where a check valve is used for purposes, other than to separate two sources such as city and mill supplies, there would be no actual loss of water but the water would not be held at the point where it is needed and extra pumping may be required to obtain necessary results. Of the same character is the leaking valve on a division line between two pressure areas. While the water is not actually lost when it passes into the lower area, the

power used to put the water into the higher area is wasted, pressures are reduced in that area and sometimes unduly increased in the lower section. A common source of loss of water is the blow-off left open at a dead end to prevent stagnation or to keep the pipe clear of sediment, or prevent freezing.

One of the most common sources of loss in a distribution system may be grouped under the heading of house waste and fixture leakage. This covers not only leaks in fixtures, but also the wilful waste where water is allowed to run to prevent pipes from freezing or to keep food cool. Such leaks are open to view and it is not necessary to mention the varieties found. One of the worst offenders in the past has been the "antifreeze" or hopper closet, which fortunately is gradually being done away with. A more recent source of waste, that is, waste which concerns the distribution system, is the gas refrigerator which requires a constant small stream of water which may be too small to register on the meter.

Leakage in reservoirs and stand-pipes is due to either evaporation or defective construction. Leakage may occur in many ways. In large reservoirs water may find its way from some point in the reservoir to an underground channel whose outlet may be miles away. Leakage or seepage through earth dams and along outlet pipes are not infrequent. A small leak of this type may seal itself off but a large leak will tend to wash out the dam. Leakage and seepage through concrete dams and reservoir walls are not uncommon and these leaks may vary from a damp spot or a trickle of water on the face of the wall to a jet shooting from the face.

METERS UNDER-REGISTRATION

Meter under-registration while not a loss of water is a loss of revenue and for this reason meters, especially large ones, should be watched closely. An oversized meter will almost always under-register on small flows and an under-sized meter will wear out more quickly than a correctedly sized meter. The compound meter takes care of the small flows but if its high side is too small to handle the usual flow, the meter wears out quickly. The location of a meter near bends in the pipe affect the accuracy of registration so that a loss may result. With proper size and suitable location, accuracy then depends on the attention which is given the meter. With small domestic meters there are other angles to be considered. While it is ideal to have all meters register 100 percent, it usually is not prac-

ticable to do so. Regular wholesale testing of small meters is conducted in some systems, but there is a point beyond which this testing costs more than increased revenue produced. While wholesale meter testing may be done, there will always be a certain number to be tested as the individual meters are found at fault and their repair will more than pay for the expense and increase revenue. However where long meter reading periods are customary, it is possible for a meter to stop early in the period and thus pass a considerable amount of water without registration. A slowing down of a meter is difficult to detect and a meter may run many months without indicating this defect. If a leak in the fixtures runs quietly on such a service, the consumer has a bill entirely out of line with his previous charges. Then in some Departments where such a practise is permitted, a rebate is made on the bill. These rebates are as much losses of water as if they were underground leaks.

UNAUTHORIZED USE

Another loss of water is caused by unauthorized use. This may vary from "dripping" through meters which do not register the slight flow thus created, to the deliberate steal by which millions of gallons of water are taken without payment through surreptitious connections to the mains or to some obscure unmetered fire-line. A common method of stealing, very difficult to detect unless caught in the act, is the use of fire-hose leading to a tank from an unmetered fire hydrant. The use of fire hose from unmetered hydrants for wetting down mill-yards in hot and dry weather is a common practice, but since this use is short and occasional, the loss ordinarily is not serious.

Another form of the unauthorized use of water is caused by the by-passing of a meter. Where the meter is set inside the plant, many opportunities are present so that a by-pass may be installed which will be difficult to locate. In some forms of compound meters where the small flow is on a by-pass, the by-pass valve is sometimes found closed, thus causing the large flow meter to pass considerable water without registration.

A deliberate steal is sometimes caused by tampering with the mechanism of the meter itself. The seal may be broken, the gear train removed and turned back; or even left out until the meter reader is expected. Gear teeth are sometimes filed so the gears do not mesh properly or the spindles are bent to accomplish the same purpose. One enterprising consumer purchased from the meter

manufacturer a dial reading only one-tenth of the correct amount and replaced the original dial.

Another unauthorized use might not be so recognized by many of us. If we see a pail or tub under a dripping faucet in a house, we might think the owner was careless about his fixtures and would not recognize it as a means of getting water without paying for it. Such is often the case, however, especially in Chinese laundries whose owners have learned that a little stream will pass a meter without registration. While this practise may exist in isolated cases, there are instances where communities, fully metered, obtained most of their water free to the great distress of the Water Department. Such was the case in a City where water was scarce and very expensive. Every householder had a barrel, with a faucet dripping into it day and night, from which they dipped water with a bucket for ordinary uses. One unusually bright citizen erected a tank, ten feet above the ground, fed it from a dripping faucet and thus was able to sprinkle the lawn without cost to himself.

Improperly regulated sewer flush tanks are a common cause of loss of water which might be classified as unauthorized use.

Finally but by no means of least importance, is the loss caused by pump slip. In reciprocating pumps, slip is caused by broken, ill-fitting or misplaced valves, and in centrifugal pumps, it is caused by the wearing of the runner. Careful pump tests, easily made, show slips of 15 percent, or more are very common, and that the pump needs overhauling.

DISCUSSION

HOWARD S. MORSE:[2] The paper of A. E. Skinner lists and describes very completely the numerous ways in which water may be wasted or lost, in the sense that in any water system there is always a considerable portion of the water produced and flowing into the distribution system which is not delivered to consumers.

Rather than continue a general discussion of this subject, the most useful contribution that I can make is to give the recent experience of the Indianapolis Water Company in attempting to account for all of the water produced and pumped.

On December 31, 1931, the task of meterizing all of the customers of the Indianapolis Water Company was completed. So during the

[2] General Manager, Indianapolis Water Company, Indianapolis, Ind.

first three months of the year 1932 our customers, insofar as they may be meterable, have been on a metered basis and the records are available for this three months period. Undoubtedly a full year's experience will be more valuable but we will take this experience of three months for what it may be worth.

Of the total quantity of water pumped during the first three months of 1932, 41.6 percent was registered on domestic meters, 18.7 percent on commercial meters, 24.9 percent on industrial meters, and 0.5 percent on public meters, or a total of 85.7 percent of the water pumped was registered and sold through meters in customers properties.

This leaves 14.3 percent of unmetered water to be accounted for. Incidentally, the output of all excepting one of the pumps of the Indianapolis Water Company is metered, and the output of that remaining pump has been carefully checked so that the figures on pumpage output are considered as about as accurate as could be obtained.

In accounting for the 14.3 percent of pumpage not sold through customers meters, it is estimated that 1.8 percent of the total pumpage is used by the City government in flushing sewers, extinguishing fires, through public drinking fountains, flushing around the City Market and Fire Engine Houses, and in cleaning streets. However, no allowance is made in these figures for water used in cleaning streets, as none of this work was done during the three months under consideration.

Allowance is also made for 0.4 percent of the total water pumped for flushing of dead end mains and through bleeders on dead end mains. This allowance is based on a record of actual flushings and check meter measurements of bleeders.

Further allowance of 6.0 percent of the total water pumped is made for under-registration of meters and unregistered water passing through meters. Accumulated tests of meters taken from service indicate that 2 per cent is a conservative allowance for under-registration of meters and the additional 4 percent covers unregistered water passing through meters.

The fact that allowance must be made for unregistered water passing through meters is emphasized in Mr. Skinner's paper, in which he mentions the fact that water "dripping" from faucets is so slight that it is not registered on the meter, and again his paper mentions a possibility of the customer taking advantage of this fact to

obtain water without paying for it. While we have had no experience with such customers, there is no question that such theft of water is possible, and several months ago the Chief Engineer and Manager of the Shanghai (China) Water Works Company, Ltd., advised me that this was a considerable problem with his Company.

As a check on this point we made a brief test in our Meter Laboratory and found that a small "drip" from a faucet running about one drop per second wasted 4 gallons per day. A large "drip" running at a rate of just less than a continuous stream wasted 75 gallons per day. Further, a "drip" that was in the judgment of several of our meter readers and inspectors an average of the "dripping" faucets encountered by them throughout the city, wasted 15 gallons per day. "Drips" of this sort are not recorded even by the best of disc piston type meters.

Using the average "drip" of 15 gallons per day, it is not unreasonable to assume that with our 72,000 customers there would be an unregistered use of water passing through meters, of 1,125,000 gallons daily, or 102,375,000 gallons during the quarter of a year under consideration.

Because of the fact that any estimate of "lost" water through "dripping" faucets is necessarily a rough guess, no further allowance is made for unregistered water passing through meters because of broken meters. However, during the three months under consideration, we removed from service 523 non-registering meters of all sizes. We read meters monthly, so in our case loss through broken meters cannot run more than thirty days.

A further allowance of 2.9 percent of the total water pumped is made for ordinary leakage from water mains and customers service lines. In arriving at this allowance, we have used a leakage from mains of 100 gallons per day per inch mile of main and $2\frac{1}{2}$ gallons per day for each service line.

Summarizing these figures, 85.7 percent of the total water pumped is accounted for through customers meters and of the remaining 14.3 percent, 11.1 percent is accounted for in the manner indicated, leaving 3.2 percent of the total water pumped altogether unaccounted for. This conclusion is almost too good to be true and we of the Indianapolis Water Company are looking forward with much interest to the results of a similar analysis of the performance of our system for the full year of 1932.

UNDERGROUND WASTE DETECTION AS A FACTOR IN THE NEW YORK WATER CONSERVATION PROGRAM

By Fred B. Nelson[1]

The aim of this paper is to present the results of a preliminary underground waste detection survey illustrating the water saving that may be accomplished in short time by simple direct methods. New York City's water consumption during 1929 averaged 880 and 930 million gallons a day in 1930. The consumption curve projected would reach our estimated available supply of 1175 million gallons a day in 1938. In view of the fact that eight or ten years would be required for building the works of a major additional supply, such as the Delaware River source, and that such an additional supply would therefore be several years too late to avoid a serious water shortage, the urgency of a comprehensive conservation program was apparent and the seriousness of the situation was emphasized by the near water shortage under the 1930 drought conditions. Minor supply developments, including extension of Long Island well system and utilization of Rondout Creek source in the Catskill Mountains are planned or under way. A pumping plant for raising 180 million gallons a day from Croton Reservoir to the Catskill Aqueduct has recently been put in operation intended to reduce overflow of the Croton Reservoir and to conserve the Catskill supply.

The methods of conserving the supply and reducing the demand which utilized in combination have materially modified the upward slope of our consumption curve in the past two years include the following:

Appeals to the public through the press, radio, movies, circulars, etc.
Plumbing inspections of unmetered premises.
Reduction of pressures, especially at night time.
Restriction of street flushing by utilizing tank flushers only instead of the extravagant use direct from hydrant nozzles.
Suspension of hose permits issued to consumers.
Enlisting the interest of the employees of other departments in reporting leaks or waste.

[1] Department of Water Supply, Gas and Electricity, New York, N. Y.

Extension of metering within the present legal and charter limits.
Restriction of excessive meter use by air conditioning plants by requiring submission of plans showing the greatest possible re-circulation or plumbing use of cooling water.

and lastly, but of major importance

Intensive direct underground waste detection—the subject of this paper.

The subsurface conditions in the boroughs of Manhattan and the Bronx consuming 60 per cent of the supply to the five boroughs are especially favorable to the development of unobserved underground waste. These conditions include streets graded by rock cut and fill in which have been installed a complexity of transit subways, electric conduits, steam lines, gas mains, water mains, sewers, etc., with the excavations for these utilities backfilled with broken rock or other loose material. Under such conditions, it is seldom that a water leak develops sufficient head to appear at the surface and rock channels direct to sewers or other exits are in many cases ample to divert the flow away from basements or other observable places.

The results of one year of systematic survey and direct searching for such waste after a period of years in which waste work has had to be confined to the correction of water flows casually observed and reported, illustrate the possible accumulation of such waste.

RESULTS OF SURVEY IN NEW YORK

While such subsurface conditions are very marked in the boroughs of Manhattan and the Bronx, it is believed that the distribution systems of these two boroughs are not the only ones of the country in which such waste exists to an extent not generally realized and the results obtained in New York in the 1931 so called "Preliminary Survey" may therefore be of service in directing your more careful consideration to this factor in the maintenance of the system in which you are interested.

For 12 to 15 years past a waste detection force under engineering supervision has been utilized to take over from the repair shop force the more difficult leak location work after reasonable effort by the repair force fails to locate the source of the flow causing complaint. That waste detection force has also been used on important and essential gradient and distribution surveys to determine size and location of needed trunk mains or other improvements of service. For the work in hand the size of this force consisting of but four field

parties precluded any systematic waste survey, except that in many cases the location of the source of the flow causing complaint involves the location and repair of from one to many leaks that were unknown and would have run indefinitely before the one responsible for the condition has been corrected.

This complaint work during the past 12 years has resulted in the annual location and repair of leakage totalling from $7\frac{1}{2}$ to $18\frac{1}{2}$ m.g.d. at a cost (based upon our conservative assumption that on the average such leaks would otherwise have continued for two years) of $4.58 per million gallons saved. The cost of complaint work, however, is not significant as its real value lies in the relief and avoidance of public inconvenience, damages and serious accidents of much greater though indeterminate value.

During 1930, on request of the department, an additional waste detection force was approved, providing for four additional field parties under engineering supervision with trucks and equipment to carry on systematic surveys for the reduction and better control of underground waste. This force was organized and working January 1, 1931. The program, subject to modification as the work progressed, was in general:

1. Preliminary survey to consist of the aquaphoning of all valves and hydrants in order to locate and stop as rapidly as possible the largest and most easily detected leaks and thus secure the greatest possible saving in the shortest possible time, together with gaugings of consumption in large convenient areas for record as incidental.
2. Final aquaphone survey on services and valves and hydrants, together with more detailed gaugings in established districts.
3. Future repetition of surveys and gaugings by districts to determine economical force and frequency of survey necessary for the proper control of underground waste.

The first item of this program, the preliminary survey, was practically completed during 1931 with the results shown in table 1, including those secured by the complaint force, which participated as actively as urgent complaint and other work would permit.

The complaint force under the urgency for waste reduction stopped a total leakage of 14 m.g.d., somewhat below their maxmum record, but 60 percent over the accomplishments of the two preceding years. The survey force, free to "gun for the big game" and the greatest possible saving in short time, located and stopped a total underground leakage during 1931 of 26 m.g.d. The contrast in the character of the

work of the complaint force where obviation of damage is the main objective and that of the survey force where water saving is the sole aim is reflected, not only in the relative total saved, but also in the comparison of the average size and number of the leaks stopped by each. The complaint force stopped 204 leaks averaging 69,000 g.p.d., as against 176 leaks averaging 150,000 g.p.d. on the survey work.

The classification of the leaks stopped is interesting in the total and there is a striking similarity in the percentage in numbers of the different kinds of leaks stopped by each force.

TABLE 1

Results of 1931 underground waste detection, New York City

	COMPLAINT WORK		SURVEY WORK		TOTAL	
Leakage stopped (m.g.d.)	14.127		26.308		40.435	
Average size of leaks (m.g.d.)	0.0692		0.1495		0.1225	
	NUMBER	PERCENT	NUMBER	PERCENT	NUMBER	PERCENT
Classification of leaks:						
Total number of leaks	204	100	176	100	380	100
Live services	82	40	78	44	160	42
Joints in main	77	38	66	38	143	38
Abandoned services	17	8	14	8	31	8
Valve leaks	18	9	9	5	27	7
Broken mains	5	2.5	5	3	10	3
Miscellaneous	5	2.5	4	2	9	2

Cost of survey work assuming that leaks found would otherwise have continued for two years	$2.42 per mg. saved
Cost of waste detection surveys	$46,800.00 (75 percent)
Cost of other work-gaugings, etc.	15,400.00 (25 percent)
Total cost (special survey force only)	$62,200.00(100 percent)

The cost of the direct survey work exclusive of gaugings of consumption for record amounts to $2.42 per million gallons saved on the assumption which we believe from experience to be conservative that such leakage without this work would have continued on an average of two years.

I will anticipate a very natural question as to how the size of these leaks and their total is determined without gaugings and in the majority of cases without direct measurement. It is felt that in general the

percentage reduction in consumption by even very effective underground waste detection in comparison with the normal percentage variation in daily consumption renders the before-and-after gauging method frequently unreliable.

Computing the amount of leakage

In determining the results here presented the amount of each leak is reported for record, together with the data on which it was computed. A small proportion in numbers, including in general the smallest leaks, are estimated using a mental "yard stick" acquired by observation of similar leaks under test. In all cases with somebody's reputation for veracity, as well as skill, at stake on the check-up, the computation or estimate must be made conservative and in some cases computed amounts have been cut in half as not conforming on inspection with the mental "yardstick" of the observer. The methods used in computing, varied and supplemented to suit individual field conditions, include the following:

1. Where the leak is actually uncovered and shut off, the minimum clear dimensions of the opening are taken, and using hydrant pressure, the orifice formula is applied with a coefficient established by test of from 0.80 for openings simulating ordinary orifices to 0.60 for elongated rough or sharp edged cracks or openings in which the stream area is more greatly reduced by contraction.

2. If the leak is of sufficient volume to produce a sensitive gradient or pressure loss when fed through a conveniently isolated small main the quantity can be closely determined in that way.

3. In the case of large leaks it may be possible to determine the pressure in the system with the leak running and with the leak shut off and afterwards measuring with a pitot tube the discharge from a hydrant nearest the leak adjusted to produce the same pressure drop.

4. Leaks on services that are not uncovered can be closely determined by using the difference of pressure head on the main and inside the building, together with the distance from the main to the approximate location of the leak. This pressure loss on the particular combination of tap and service size represents very closely the amount of the leak.

All these methods have been checked by tests and whenever feasible the actual break in such as service pipes is taken to our testing station and the discharge measured by weight and stop watch to check and aid the proficiency of the computer. In a recent check-up on 12

such leaks the total of the estimates was within 2 percent of the actual. Recent pitometer measurements to check field computations of 3 large leaks showed the computed total to be 8.5 percent below the measured flow.

The morals to be drawn from the story I have outlined (which I hope is not accepted as in the "Fairy" or "Bed Time" class) are:

(1) Do not be too positive that your system is tight, based on the fact that your complaints are few or on the assumption that the leaks that have come to the surface begging for attention are the only ones you have.

(2) Underground waste surveys should be incorporated in regular maintenance procedure, the size of the necessary force and the economical frequency of such surveys to be adjusted to suit local conditions.

DISCUSSION

THOMAS J. SKINKER:[2] During the past six years extensive Pitometer surveys have been made upon the St. Louis Water Distribution System to curb leakage. House waste and fixture leakage is our greatest source of trouble. It is usually found in second and third class residential sections, multiple residences, slum districts and industrial sites.

During the extremely cold periods this waste is greatly increased due to faucets left open to prevent water in the pipes from freezing. There are a great number of cases where the property owners have instructed tenants to let water run.

Prior to the drouth of 1930 all of our peak loads occurred during extreme cold periods.

During the years 1927 to 1929, the following amounts of leakage were found:

Underground leakage equal to 0.9 m.g.d.

House waste leakage equal to 7.2 m.g.d.

Loss due to meter under-registration, 4.1 m.g.d.

Loss through pressure zone left open, 8.1 m.g.d.

(This is not a direct loss, but energy wasted from one system to another.)

The number of leaking fixtures found was 12,300.

During 1930, 9,658 fixture, leaks were found, approximately 5.6 m.g.d. Underground service leaks totalled 2.3 m.g.d.

[2] Engineer in Charge of Distribution, St. Louis, Mo.

During 1931, 6,803 fixture leaks were found, approximately 3.9 m.g.d.; and 1.3 m.g.d. of underground leakage.

The house waste in St. Louis is equivalent to 6 to 7 percent of the total consumption, and the underground leakage to less than 1 percent. This excessive amount in house waste is principally due to the fact that St. Louis is only 9.7 per cent metered.

The negligible amount of underground leakage found is due to the fact that the soil in St. Louis is of a more or less impervious clay which causes leaks to come to the surface.

Those districts which have shown an excessive amount of leakage have been divided in four groups according to the amounts of leakage and the frequency of occurrence. They are surveyed in periods of six months to a year, and immediately followed up by a very rigid inspection for service and fixture leaks by a crew of trained inspectors. The leaks as found by these inspectors are tabulated upon index cards for each house and made a permanent record in our files. When these records have shown that certain flat rate services have had a frequent number of leaks within a short period of time, it is possible for the Water Commissioner to immediately order the installation of meters upon such services.

This policy was adopted in lieu of the selective metering program which was instituted in 1929 and which met with antagonism from the Real Estate Exchange, business organizations and property owners in general.

This conservation program has had a material effect upon our average daily consumption, which has been reduced from a peak of 123 million gallons in 1929 to 116 million gallons in 1932.

There is only one way to eliminate house waste and that is to meter and let the consumer pay for all the water he uses and wastes.

In the curbing of underground leakage the following precautions are beneficial:

For defective pipe: Rigid inspection and pressure test at foundry—use of steel pipe on large size trunk mains, and longer lengths of cast iron pipe to cut down on number of joints required.

For defective joints: On lead joints, pneumatic caulking; use of non-caulking materials; use of latest type of pipe couplings.

In laying pipe: rigid inspection and proper supporting of pipe in the vicinity of other utilities—such as sewer, gas and electric conduits.

Electrolysis: Coöperation with local utilities in the installation of negative return systems to source of supply.

Corrosion: In suspected localities back fill trench with chat or other neutralizing agents.

Meter waste: Rigid and frequent tests by means of the Pitometer on all meters in place three inches and larger; and regular periodic tests and cleaning of small meters.

WILLIAM W. BRUSH:[3] You have been told of the amount of leakage that we have located and stopped in New York and you may get the impression that our system is a very leaky system.

On the other hand, the men who have to do with very extensive excavations below the surface of the streets such as the rapid transit subway builders all state that the water system is generally in very good condition as far as leaks are concerned. The number of leaks we find are small compared with our six hundred thousand services and two million pipe joints. One leak in a small system may very seriously increase the demand and several thousand leaks on New York City's system represents millions of gallons of water wasted daily. The method to be followed in locating and stopping these leaks depends upon local conditions which may indicate the desirability of using the pitometer, aquaphone, hydrant hose and meter or some other method.

Whatever the method selected it is quite evidently a service to the community to locate and stop a leak that exists outside of a building. That leak is doing absolutely no good to anybody and may prove to be a source of danger due to the fact that the softening of the ground in the vicinity of the leak will probably cause ultimate damage. One of our superintendents who was an excellent pipe man told me before he died a few years ago that he thought that he could trace back practically all of the large breaks to leaks that probably had existed for months or even years, before the break occurred. We should, therefore, in the interest of the communities we serve make surveys of our distribution system, if we have any suspicion at all that there is a possibility that leaks exist.

[3] Chief Engineer of Water Supply, Department of Water Supply, Gas and Electricity, New York, N. Y.

BREAKS IN CAST IRON PIPE GRIDIRON SYSTEMS

By W. C. Mabee[1]

During the past 35 years a record of breaks in water mains in the gridiron system of the Indianapolis Water Company has been kept by one man, who is still carrying on. I refer to the emergency repair man, who has, at all times, been sufficiently interested in his job to keep a diary of breaks and their causes. During all these years this man has been and still is attached to the maintenance division, and the information presented is therefore quite reliable, although there is no certainty that all breaks were reported.

I have analyzed these data and have classified them into eleven groups of causes as follows:

1. Settlement
2. Electrolysis
3. Increased Pressure
4. Vibration
5. Improper Care by Others
6. Defective Pipe
7. Freezing
8. External Blow
9. Flood
10. Water Hammer
11. Miscellaneous and Unknown Causes

These general groups have been sub-divided as follows:

Settlement—Group 1 (26 per cent of breaks):

(a) Unequal bearing on soil resulting in settlement, pipe strain and failure. The evidence of this sort of rupture is usually described as a clean break, meaning that the pipe has snapped in two.

(b) Improper backfilling of sewer trenches lying adjacent to or crossing beneath water pipes induces settlement under the water main. This cause is readily established when the facts are known concerning the history of the sewer installation.

(c) Improper backfilling behind building walls adjacent to water pipe resulting in settlement of the pipe and consequent failure.

[1] Chief Engineer, Indianapolis Water Company, Indianapolis, Ind.

TABLE 1

Causes for 222 breaks in water mains during the past 35 years in the city of Indianapolis, Indiana,—1897 to 1931 inclusive

	NUMBER OF BREAKS		PERCENT
1. Settlement:			
a. Unequal bearing on soil resulting in settlement and pipe strain	17		
b. Settlement of sewer trenches alongside water mains or crossing under water mains	21		
c. Settlement of backfill behind walls adjacent to pipe lines	3		
d. Caving basement walls causing movement of pipe	2		
e. Broken sewers under water pipe causing settlement	2		
	2		
f. Settlement of walls or abutments through which pipe passes	2		
g. Temporary shores placed under pipe and subsequent settling	5		
h. Settlement of pipe disturbed by conduit construction	6	58	26.1
2. Electrolysis		68	30.6
3. Increased pressure (fire pressure)		16	7.2
4. Vibration:			
a. Vibration from moving trains may result in breaks	0		
b. Vibration from street railway and truck traffic caused by rigid bearings of conduits, manholes, track beds or other structures resting directly on water pipe	16	16	7.2
5. Improper care by others (after pipe is exposed)		15	6.8
6. Defective pipe:			
a. Faulty castings	6		
b. Pipe with thin walls	2		
c. Pipe cracked in handling	5	13	5.9
7. Freezing:			
a. In ground	8		
b. On bridges	1	9	4.0
8. External blow:			
a. From excavating machinery	6		
b. From pile driver	2	8	3.6
9. Flood:			
a. Washouts	9		
b. Dredging	1		
c. Collapse of bridges	0	10	4.5
10. Water hammer		4	1.8
11. Miscellaneous and unknown (including soil corrosion)		5	2.3
Total		222	100.0

(d) Caving basement walls. We have had several instances of this sort of failure.

(e) Broken sewers under water pipe undermining the same.

(f) Settlement of abutments or walls surrounding pipe laid prior to the construction of these walls and without proper cushioning provisions causes the pipe to fail in shear.

(g) Rigid bearings of pipe on temporary shores left in place by building contractors or by other utilities. Eventually these shores collapse or produce unequal settlement and failure results from pipe strain.

(h) Settling of pipe where the pipe bed has been disturbed by the installation of conduit construction. This latter case may also be attributed to the classification entitled "Improper Care by Others."

Electrolysis—Group 2 (30 per cent of breaks):

Leaks from electrolysis have been frequent in Indianapolis. They are caused by stray electric currents originating from street railway operation. The destruction is slow but sure. Leaks occur in those sections of the system known as positive areas where the stray current leaves the water pipe, returning to the rails or other metallic structures. Breaks occur usually under car tracks or adjacent to rails. Water pipe affected by electrolysis may survive for a long time, but will finally fail under increased pressure. Leaks from electrolysis in Indianapolis are less frequent today, because of the installation of a number of sub-stations by the Street Railway Company, and also by the application of mitigation taps and electrolytic drainage in the vicinity of these sub-stations.

Increased Pressure—Group 3 (7 per cent of breaks):

Raising fire pressure is also a potential menace to water pipe gridiron systems. This practice develops weaknesses which frequently result in pipe failures.

Vibration—Group 4 (7 per cent of breaks):

(a) Water mains may break under tracks from vibration caused by moving trains where pipes are not properly protected.

(b) Vibration from street cars and street traffic in places where there are rigid bearings on water pipe, such as masonry conduits, manholes, or other utility structures, and at points where no earth cushion is present to protect it frequently results in broken pipe.

Improper Care By Others—Group 5 (7 per cent of breaks):

Lack of caution in caring for exposed water pipe adjacent to buildings, sewers, bridges or other construction work has contributed to breaks.

TABLE 2

Thirty-five year record of breaks in cast iron gridiron system classified as to cause

Indianapolis Water Company

YEAR	SETTLEMENT	ELECTROLYSIS	INCREASED PRESSURE	VIBRATION	IMPROPER CARE BY OTHERS	DEFECTIVE PIPE	FREEZING	EXTERNAL BLOW	FLOOD	WATER-HAMMER	UNKNOWN AND MISCELLANEOUS	TOTAL	BREAKS IN 5 YEARS	MILES OF PIPE	BREAKS PER 100 MILES PER YEAR
	1	2	3	4	5	6	7	8	9	10	11				
1897			1									1			
8	1											1	5	200	0.5
9															
1900						1						1			
1	1			1								2			
2	1											1			
3	1								1			2			
4	1		1			1			5			8	24	252	1.9
5	1		3		2	1					1	8			
6	1	1			1					2		5			
7	1		1									2			
8	2	1	1									4			
9	4		2		1				1		1	9	27	303	1.8
1910	1	1	1				1					4			
11	1		1	1	5							8			
12	2	1	1	2			1			1		8			
13	3			1					1			5			
14	2	4			1	1						8	39	380	2.0
15	1	4				4						9			
16	2	4		1		1		1				9			
17	4	6			1							11			
18		3		1			7					11			
19	1	2	2	1	1							7	46	430	2.1
1920		5		1				1				7			
21	3	5	1		1							10			
22	4	5	1						1			11			
23	1	2				1		1		1		6			
24	1	5		1	1							8	42	508	1.6
25	2	6						1			1	10			
26	2	4		1								7			

TABLE 2—*Concluded*

YEAR	SETTLEMENT	ELECTROLYSIS	INCREASED PRESSURE	VIBRATION	IMPROPER CARE BY OTHERS	DEFECTIVE PIPE	FREEZING	EXTERNAL BLOW	FLOOD	WATER-HAMMER	UNKNOWN AND MISCELLANEOUS	TOTAL	BREAKS IN 5 YEARS	MILES OF PIPE	BREAKS PER 100 MILES PER YEAR
	1	2	3	4	5	6	7	8	9	10	11				
27	3	4		1		1						9			
28	5	5		2		2		2			1	17			
29	3			1								4	39	630	1.2
1930	3			1	1			2			1	8			
31									1			1			
Total.....	58	68	16	16	15	13	9	8	10	4	5	222		654	1.0
Percent.....	26.1	30.6	7.2	7.2	6.8	5.9	4.0	3.6	4.5	1.8	2.3	100	Avg.	386	1.74

Defective Pipe—Group 6 (6 per cent of breaks):

(a) Faults in pipes resulting from improper materials in the castings evidenced by honeycomb after a break.

(b) Pipe cast on side and having thin wall on one side.

(c) Pipe cracked in transit or in handling and laid without detecting flaws. Where damage of this sort is slight, pipe may survive a long time but will ultimately fail under some undue stress.

Freezing—Group 7 (4 per cent of breaks):

(a) Pipe laid in trench too shallow for the climate, may during an unusually cold winter, freeze and burst.

(b) Pipes laid on bridges in cold climates without proper insulation or with poor circulation and low velocity are very liable to freeze and burst.

External Blow—Group 8 (4 per cent of breaks):

(a) Careless handling of steam shovels or digging machines has frequently resulted in broken pipes. Accidents of this sort may occur where proper care is exercised but the presence of a water pipe is unknown to the operator.

(b) We know of several instances where pipes have been broken by the act of driving piles.

Flood—Group 9 (5 per cent of breaks):

(a) Stream crossings not properly protected have been undermined by floods and washed out.

(b) Stream crossings have failed from dredging operations which have removed the pipe support, causing disastrous settlement.

(c) Floods have caused bridges to collapse, resulting in broken water pipe.

Water Hammer—Group 10 (2 per cent of breaks):

(a) Water hammer induced by surging in the mains is always a potential source of danger and may be brought about by closing valves too fast or by the operation of quick closing valves. We have a record of several such failures, notably a quick closing valve supplying water for locomotive tanks.

Miscellaneous and Unknown Causes—Group 11 (2 per cent of breaks):

Under this group I have classified a failure from a split plug. This casting was not defective, but was not properly braced in the fitting. Breaks from unknown causes and from other miscellaneous causes, such as soil corrosion, are included in this classification.

Although I have outlined eleven classifications for breaks and have expanded these into twenty-five contributing causes for convenience in listing, there appears to be but five distinct causes for pipe failure; i.e.,

A. Pipe Strain Breaks, usually from settlement, including the eight subdivisions listed under classification (1) and may also include classification (5) improper care by others, and (9) flood failures.
B. Breaks resulting from corrosion, including electrolysis and soil corrosion.
C. Internal Pressure Failures, including classification (3) raising fire pressure, classification (10) water hammer and (7) freezing.
D. External Pressure Failures, including classification (4) vibration from trains or street traffic and classification (8) external blow.
E. Defective Pipe Failures.

LONGEVITY OF PIPE

We can discover no tendency for breaks in water pipes to become more frequent, because of the age of the pipe. In Indianapolis 8 percent of the pipe lines are between 50 and 60 years old, 47 percent less than 20 years and 15 percent are less than 6 years old.

CONDITION OF PIPE

Our investigation of pipes of all ages removed for one cause or another indicates that there has been no deterioration of the metal itself, electrolysis excepted.

FREQUENCY OF BREAKS

While 222 breaks in water mains may appear to be many, it should be borne in mind that this record is over a period of 35 years, they

represent on the average 1.74 breaks for each one hundred miles of mains per year; furthermore, it is probable that breaks from electrolysis will become less frequent, because of recent changes in the electrical railway system. If, in the comparison of percentages of breaks to miles of mains, we were to eliminate the breaks from electrolysis, the number of breaks per one hundred miles of pipe would be reduced from 1.74 to 1.22 breaks per one hundred miles per year.

It will be noted from this discussion that the great majority of breaks in water mains have occurred from causes beyond the control of the water plant operator. Greater care and more caution on the part of others must be exercised if the number of breaks are to be reduced. Very few failures may be attributed to defective pipe, especially if all pipe is tested before shipment, after shipment and before it is laid.

Rigid inspection of all pipe laid and care to see that the pipe is properly bedded will result in minimizing the number of preventable breaks from settlement.

It is bad practice to permit shoring to be left under pipe, or to permit structures to rest upon pipe.

Coöperation between the public utilities themselves, with respect to the proper care of each other interests, will also tend to reduce trouble.

The remedy for frozen pipe lines is self evident. Floods can not be prevented, but precautions can be taken to reduce the hazard.

Having exercised all of these precautions the water department will have done its full duty and may expect preventable breaks to be reduced to a minimum; however, contractors will continue to be careless, accidents will happen and the superintendent will still be called upon to repair mains broken from causes beyond his control.

DISCUSSION

THOMAS F. WOLFE:[2] Of all materials of construction, cast iron pipe is one of the few that goes into a type of structure subject to many vicissitudes that are not evident when the original installation is made. Ordinarily when we build a bridge or a building, we carefully design the foundations to withstand the loads they are to carry. In the ordinary course of events, these foundations are never disturbed but occasionally either of two things may happen. Another structure

[2] Research Engineer, Cast Iron Pipe Research Association, Chicago, Ill.

may be built adjacent to our bridge or our building and so close to it that our foundations may be affected. In this case, by underpinning or other methods, we protect our own structure and no damage results. The other possibility is that due to causes beyond our control, such as freshets, earthquakes or similar natural phenomena, our foundations are disturbed without our having a chance to protect them. In this latter case, there is great danger that our structure may fail. If it fails, we ordinarily ascribe the failure to the inability of the foundations to withstand the influences that were brought to bear on them. This explanation is very easy for the layman to understand and no one would be inclined to condemn the material of which the building or bridge is constructed for the failure.

On the other hand, let us consider what happens when a pipe in a gridiron system fails. These failures occasionally are spectacular and as a general rule, result in some inconvenience to the citizens. The pipe itself is a prosaic thing and without giving the matter much thought, newspapers and laymen jump to the conclusion that the fault lies with the pipe. Let us for a moment, consider the different things that may have happened since the pipe was laid. First of all, many of the breaks that take place occur in pipe that may have an age of anywhere from thirty to seventy-five years. Pipes made half a century ago were manufactured in the light of what was then known of metallurgy and foundry practice. The art of laying pipe was then new and experience had not yet taught waterworks men that certain precautions are necessary to insure the safety of the pipe line. Furthermore, the heavy loads of present day street traffic were not foreseen, nor were the complicated systems of underground utilities thought of. The same pipe that at one time lay in a street where the heaviest load was a slow-moving horse-drawn vehicle and where no other utility occupied the street, may now be serving in a heavy traffic street subjected to the pounding loads of ten ton trucks passing at high speeds and at frequent intervals. Furthermore, it may have been disturbed for the purpose of installing gas mains, sewers, electric light, telephone and telegraph conduits, steam lines, police and fire telegraph lines, pneumatic tubes, subways and the other types of utilities now occupying our streets. In spite of these changed conditions, when this veteran pipe fails, the particular features of it that interest the newspaper men consist only in the fact that it has failed. The history of the pipe line often is not known at the time of the failure, nor is it spectacular enough to make news.

On the other hand, the waterworks man or the gas man in ascribing a cause to the break, will usually develop all the information pertaining to it and as a rule be satisfied that the cause does not lie within the pipe itself but rather in the abuses to which the pipe has been put.

An investigation of the causes of breaks in a number of properties throughout the country and by a number of disinterested observers, shows the causes to be as follows in the order of their importance:

1. Settlement
2. Poor Construction Practice
3. Temperature Changes
4. Vibration and Fatigue
5. Pressure Changes
6. Defective Material
7. Electrolysis

Under the first heading, "Settlement," is included the settlement of ditches in which the pipe is originally laid, settlement due to the construction of adjacent structures, the settling of other structures on the pipe itself such as conduits, sewers, subways, retaining walls and other things. The remedy for this type of failure lies, first, in the proper preparation of the ditch that will receive the pipe and in proper backfilling. By proper preparation is meant the avoidance wherever possible of disturbing the earth below the bottom of the pipe. The one exception to the rule is that where rock is encountered, an excess of at least 10 inches in depth should be provided and the bottom of the ditch filled with loose earth properly tamped before the pipe is laid. After the pipe has been laid, backfilling should receive as careful attention as any other part of the pipe construction job. Loose material only should be used and this material should be thoroughly tamped under the pipe at least half way up the diameter of the pipe. It is important to keep in mind that the backfilling underneath the pipe should be tamped. Very often, due to the necessity for keeping ditches open until a test is made, there is more or less caving of banks with the result that some loose material gets under the pipe before backfilling starts. There is some danger that this part of the backfill will remain loose.

The settlement of pipe due to the construction of other underground utilities can only be avoided by exercising proper care during the construction of these utilities. Precautions should be taken to see that the pipe is properly braced, that parallel trenches are properly sheeted and that other structures are so built that when they in turn

settle, they will not rest on the pipe. The practice of protecting existing lines from damage during construction is quite common among gas companies, but unfortunately many water departments do not exercise the same care as the gas companies in this regard.

Under the heading, "Poor Construction Practice" is included faulty handling of pipe, faulty design and poor judgment in carrying out construction of either the line itself or adjacent underground utilities. In the handling of the pipe from the time it is shaken out of the mould in the foundry, until it is placed on the cars, the pipe companies use every precaution to avoid damage to their product. Pipes are so loaded in the cars as to avoid as much as possible, any damage during transit. After all these precautions are taken, however, the contractor very often handles the pipe as though it were impossible to damage it. Pipes should never be dropped on hard pavements and it is a good rule to follow that it should not be dropped under any circumstances. The additional effort necessary to properly string the pipe along the side of the trench costs very little and is excellent insurance against breaks. Before each length of pipe is placed in the trench, it should be inspected for cracks. It is much simpler to cut off the cracked end of the pipe before installing it in the line than it is after installation.

When installing pipe in cool weather, they should not be set all the way home. This is particularly true of gas lines and of lines carrying water that varies widely in temperature during the year. While, as a general rule, the pipe and joint are capable of absorbing the stresses due to the ordinary changes in temperature, it is highly probable that some of the unexplained breaks in gridiron systems are due to temperature stresses.

From time to time, we hear of breaks in cast iron pipe adjacent to street car lines, railway lines and subway structures. Some of these lines have been in service a great many years and there has been no noticeable change in surrounding conditions during these years. It is highly probable that such of these breaks as are not definitely due to settlement may be due to fatigue stresses. Many of these lines have insufficient cover and there is no opportunity for the earth to absorb the shocks. The remedy for this difficulty is to lay pipe as far away from these influences as possible and at a lower depth. In many parts of the country, the depth at which pipe has been laid in the past was determined by the depth to which frost penetrated. This may have been good practice many years ago, but at the present time in locating a pipe line, engineers should take into account present day street traffic and provide accordingly.

Under "Pressure Changes" is included additional pressure caused by water hammer as well as changes in pressure conditions since the pipe was installed. Pipe ordinarily used in gridiron systems is capable of withstanding a much higher pressure than that at which it is rated. However, the rapid shutting of valves or fire hydrants or the sudden stoppage of use of water after a fire, may result in water hammer developing that raises the pressure to a figure that approaches the danger point. These additional pressures will search out the weak point of the pipe line which may be a point that is already stressed to an excessive amount because of settlement, temperature changes or other influences. In many systems, due to a change in the source of supply or method of distribution, pipes that at one time were required to carry water at a very low head are now required to withstand higher heads. Here also the combination of the greater pressure with other stresses may result in a fracture of the pipe.

Among the causes of failure, we naturally find defective material. Every effort is made by the pipe manufacturers to produce good material and every effort is made by the pipe purchaser to inspect this material to see that it meets his specifications. None of us is perfect. Consequently, from time to time, some imperfect material will show up. The best remedy for faults, due to imperfect material, lies in a rigid test of the completed line at a pressure in excess of the pressure under which it will be called upon to work. The pipe should be kept under this pressure for several hous so that any weak points or inferior material will have an opportunity to fail and be replaced before the line is actually put into service.

Electrolysis as a reason for failures in gridiron systems has, during recent years, become comparatively rare and when experienced, can only be taken care of by proper precautions in handling the return current in street car systems.

Before leaving the subject of breaks, a few remarks regarding the frequency of breaks is not out of order. The number of breaks to be expected in a distribution system depends largely on local conditions. In cities where the underground system is continually disturbed, it is reasonable to suppose that more breaks will be experienced than in smaller towns where such is not the case. It is the practice in Detroit to keep a record of the number of breaks in the gridiron system. The Water Department report for the year ending June 30, 1929, states that 121 breaks were experienced during that year. Their system at that time comprised 2,757 miles of main. This means that

they had 0.044 of a break per mile of pipe or to put it another way, there would have been one break in each 120,000 feet of pipe. During the year ending December 31, 1931, the city of Chicago experienced 142 breaks in 3,692 miles of pipe. This is equivalent to 0.038 of a break per mile of pipe or one break in each 137,000 feet of the system. The Citizens Gas Company of Indianapolis experienced 290 breaks in a period of 16 years. This system consists of 368 miles of pipe. Assuming that the average mileage of pipe during the 16 year period was 350, we find that they experienced 0.052 of a break per mile of pipe per year or one break per year in each 102,000 feet. This figure is larger than either that of Detroit or Chicago due principally to the fact that many of these breaks were in 3-inch pipe, a size not ordinarily found in water distribution systems. If we eliminate the breaks in this size, we find that one break per year was experienced in each 157,000 feet of main. In the case of the Indianapolis Water Company, 222 breaks were experienced over a period of 35 years, an average of 6.35 breaks per year in a system that now comprises 633 miles of pipe. Included in all of the breaks mentioned are cases of split pipe where the actual leak was very small and where the cost of the repair was slight. On the whole, the experience of the four properties mentioned, which is fairly representative of systems of the sizes involved, is indicative of the fact that, in spite of changed conditions under which old pipe are called to work, they still function with remarkable success. With our present knowledge of metallurgy and with modern foundry practice, we believe that present day pipe will give an even better account of itself. This will be doubly true, due to the knowledge that waterworks men have gained from past experience and because of the greater degree of skill exercised in present day water works construction.

THOMAS H. WIGGIN:[3] The papers by Mr. Wolfe and Mr. Mabee have covered the subject in a very complete way. The records kept by that conscientious employee of Mr. Mabee's Company are unusual and should be very much appreciated by the Society and others.

It is not often possible for many of us to be first-hand observers when breaks are dug up and most of us have only a limited number of breaks with which we are really acquainted. In the speaker's experience of over 35 years in pipe work—much of which experience it ought to be said has been from the construction and design side

[3] Consulting Engineer, New York, N. Y.

rather than from the maintenance side—there are a number of typical cases which stand out in memory.

1. A 36-inch pipe of moderate thickness (1.03-inch if I recall correctly) split through the bell at two places releasing a large semi-oval piece of pipe about one-third the circumference of the pipe at the bell and tapering out to a point near the middle of the pipe. The metal was good. The pressure was not over 75 pounds. A point of ledge was found projecting up from the bottom of the trench and the pipe had evidently been resting on it near one side of the break. The cause of the break was doubtless the heavy bearing on a point of rock, although why the whole side of the pipe should come out under such moderate pressure was a little difficult to see.
2. A 20-inch Y-branch split in two along the plane of the two branches beginning at the bells. This was a non-standard casting not made by a pipe foundry and was deficient in metal at the crotch. It had been recaulked several times on account of leakage into a sewer trench. The pipe was however perfectly supported over the trench by a system of steel beams and hanger rods. The only explanation was deficient strength in the casting and the addition of stresses caused by hard caulking to those already existing owing to water pressure.
3. A 10-inch Y at a pumping station in a force main carrying 250 to 300 pounds from a reciprocating pump split open in the crotch without sign of previous cracking. It was a standard 250-pound flanged fitting. The only reason for the break beyond the slight excess of pressure carried at times was the general weakness of these Y laterals, although even the weakest of them has a factor of safety of nearly two when tested. A committee of the Cast Iron Pipe Research Association has been making tests of fittings and has found the Y to be unusually weak and is making improvements.
4. A 72-inch Class D flange and spigot manhole casting with manhole very close to the flanged end split longitudinally through the flanged end and manhole during a quiescent test to the static working pressure. Analysis of the strength of the casting showed that the manhole was too near to the flange and that the section of metal was so small as to give a deficient factor of safety at that point, although there was still some factor of safety, perhaps 2. The walls of the casting were heavy and the iron of coarse grain but without sign of rust in the cracks.
5. A 36-inch by 16-inch flanged Tee of class D thickness broke in test under about 300 pounds showing it to have a factor of safety of only about 2. Computations of these castings show them to be considerably deficient in longitudinal section through the branch.
6. An 8-inch sand spun pipe of thickness about equal to the present 150-pound class split under moderate pressure considerably less than 150 pounds when the newly laid pipe was tested. The split reached neither the bell nor the spigot. No cause was assignable unless there was an incipient initial crack on the inside, not visible when the pipe was laid and not deep enough to show rust immediately after fracture.

7. Another 8-inch sand spun pipe broke recently by splitting. The fracture was not perpendicular to the wall of the pipe, but at an angle of about 30° with a jagged surface throughout as if violently torn apart. There was no initial rust in the break and the conditions of laying were unusually fine. The pressure charts showed no unusual water ram.
8. A series of breaks in a 48-inch pipe under light pressure and with a correspondingly moderate thickness, but laid under a fill of about 12 feet. This pipe, after about 20 years without breaks, split repeatedly, a length or two at a time, when the pressure was raised 20 pounds or so in 5 or 10 pound steps. The raising of pressures is not an uncommon condition under which breaks occur. In the case of large pipes under heavy earth cover, like the above it would seem reasonable to think that the combined trench loads and water pressure have the pipe well up to its proper breaking strength so that a little more pressure causes the strength to be exceeded. This however leaves most cases unexplained since similar breaks occur where earth loads are normal and factors of safety of 3 or 4 should exist even after raising of pressures.
9. A series of breaks about 1 per mile developed in a new 24-inch main about 15 miles long during first test. These were repaired and no trouble has been experienced since. Unseen initial cracks were the evident cause. Each of us can recall such occasions, more or less in number according to his experience.

Mr. Wolfe has listed well the various causes contributing to breaks. Nevertheless it is probably true that a majority of breaks are never explained in the sense of proving, by a comparison of stresses computed from known loads with known strength of metal, that the pipe was subjected to stresses beyond its ultimate strength. To illustrate, water ram is sometimes an assigned cause, but considering that even A. W. W. A. Class B cast iron pipes will stand test pressure from 1000 to 2000 pounds per square inch according to size it is not a mathematical explanation to say that an extra 25, 50 or 100 pounds from water ram caused the break. The speaker has been forced to the conclusion that most splitting breaks in pipes are due to incipient cracks not noticed at the time of laying, except that in cases of larger pipes, trench loads, that is, exterior loads, sometimes contribute to an important degree.

The Cast Iron Pipe Sectional Committee now functioning under the sponsorship of the A. W. W. A. and three other societies has been investigating the effect of trench loads by tests made at Iowa State College. These tests have shown that the effects of earth loads are very considerable, which of course all of you knew, but have gone

further in assigning mathematical values to these effects, which will permit rational design of thicknesses of pipe for all conditions of internal pressure, external loading and methods of laying. The considerable effect of the omission of tamping under pipes and also the effect of blocking under pipes in reducing strength has been shown by these experiments which the Committee expects to publish in condensed and digested form.

Among the breaks noted above with which the speaker was personally familiar were some of fittings and other similar cases have been reported to the speaker. The Sectional Committee has had destructive tests of some fittings made and the Cast Iron Pipe manufacturers have had a lot more made. At the end we have confidence that weak designs will be eliminated. Breaks in fittings have probably always been less in number even in proportion to the whole number in use than in straight pipes, yet their factor of safety is much less than that of pipe so far as structural strength of perfect castings is concerned. If this proportion of breaks is less it can only be explained so far as the speaker can see by less likelihood of damage in handling and perhaps less effect of earth load owing to the stiffening effect of an extra number of bells.

Injury to pipe before laying is more likely to occur to thin than to thick pipes. Resistance to dropping blows increases we are told by the laboratories as about the 1.5 power of thickness, i.e. a pipe of a given diameter 1.6 times as thick as another is about twice as strong against impact. Stronger metal will of course compensate for thickness, provided the strong metal is also tough.

The present period is one of experimentation in thinner pipes of generally stronger iron made by centrifugal and other special methods; also in longer pipes cast 16, 18 and 20 feet long and made still longer by welding or screwing cast lengths together. Even 2-inch pipe can be had in 18 foot rigidly assembled lengths and 6-inch up to 40 feet in welded units is being used, the speaker is informed, for gas lines. The thinning process was carried too far in the opinion of most of the pipe manufacturers and the new government specifications have come back part way toward the pit cast standards. Extra length as affecting pipe-stem or cross breaks is still a subject for consideration, but the increases to 16 feet in small sizes seem so far to have caused no unfavorable comment as to breaks. The gas companies have long used deep bells for cement joints which produce rigidity and would seem to give as severe a test as lengthening the pipe.

Mr. Wolfe's paper has indicated precautions which will tend to minimize the number of breaks. The speaker would emphasize certain of these precautions.

1. Inspection at the foundry and at the trench is the most important precaution. Eccentricity, producing thinness on one side, is still a reported cause of breaks in pipes not cast centrifugally. Incipient cracks not discovered at time of laying are in the opinion of the speaker the cause of most breaks.
2. Avoidance of careless handling in order to prevent incipient as well as visible cracks.
3. Avoidance of stones or points of rock under pipes. Blocking under pipes is also hurtful according to recent tests though it may be compensated for by using somewhat thicker pipes.
4. Ramming of backfilling under and along side of pipes. This materially increases its resistance.
5. Avoidance of excessive water ram by elimination of shut-off valves or regulating valves of types which are too quick-closing; also by use of improved controls for starting and stopping pumps. There is also room for study in adaptation of proper types of check valves.
6. Inspection of trench work done by others near or under one's pipe line. Such work results in destruction of the unyielding foundation and of the support afforded by backfilling, unless great care is exercised in temporarily supporting the pipe and in compacting the backfilling beneath it. A trench parallel with and close to a water pipe obviously removes much or all of the support which makes backfill of assistance to the pipe in resisting earth loads. Such a trench is also likely to add to the earth load because of removing the upholding power of one side of the original water pipe trench. Moreover, some settlement is almost sure to occur, although fortunately a limited amount of such settlement is not disastrous because of the flexibility of joints. There is probably too little attention paid to intelligent and forceful inspection of work done near one's pipe line by other interests.

In conclusion it should be said that there is an economic limit beyond which it does not pay to go either in thickening pipes or in increasing laying costs in order to reduce number of breaks.

MR. MABEE:[1] I would like to add one thing. As we have become more familiar with the characteristics of pipe we have reduced the factor of ignorance that was introduced into the engineering of pipe installations years ago.

Recently I had occasion to uncover a 36-inch pipe that was laid in 1897, the shell of that pipe was two and a half inches thick. We were installing a 36-inch Venturi meter, and we had to throw away a length of that extra heavy pipe because it would not match in alignment with the product of the Builders Iron Foundry for doing exactly the same pressure work.

I would just like to accent that a little bit, our factor of ignorance has been reduced, the pipe manufacturers, are more scientific, the engineers are knowing more and there is less waste of useful cast iron in our present day installations.

MEMBER: I would like to ask Mr. Mabee with his splendid record in Indianapolis, if there has been a grouping of the breaks as occurring in certain seasons.

MR. MABEE:[1] I could make a table on that, but have not done it yet. We had many breaks due to freezing one winter. There were more breaks in that winter than over the whole thirty-five year period. But I think that you will find that breaks are scattered pretty much throughout the year. The electrolysis breaks may occur at any time. Of course, they are not occurring now as frequently as they did. This winter we had a break in a 4-inch main; we tried to uncover it, the weather was desperately cold and the men worked a long time, but finally we discovered that the break was caused by settlement where concrete had been dumped on top of the pipe during building operations. From my observations I cannot see that the breaks occur at any particular time of the year.

MR. VAN LOAN (Philadelphia, Pa.): My point in bringing that question up, Mr. Chairman, is not particularly the atmospheric temperatures, for it would be more the temperature of the watermain, or the water where the temperature was fluctuating from thirty to seventy degrees, or something like that. They are the maximum and the minimum in the Spring and the Fall. That is why I brought that particular point up.

MEMBER: Mr. Mabee has stated that about 30 percent of the breaks are due to electrolysis and we all know that often that is due to the fact that the electric railways are not careful in keeping the line losses down to the minimum, I am just wondering whether or not at Indianapolis there has been any attempt made to have the parties responsible for the damage done by electrolysis pay the charges they cause? It seems to me that that would be one way of getting some coöperation from the parties concerned and they would be more interested in keeping their line interference down to a minimum.

CHAIRMAN GIBSON: I think that the automobile is curing a great many of our electrolysis troubles, from the trolley road, anyway. Mr. Mabee can probably answer that for they have trolleys in Indianapolis.

MR. MABEE: It has been a rather up-hill fight to get the railway utility to do anything. They will admit they are the cause of electrolysis. So in Indianapolis we have had to suffer for a long time, until finally, as Mr. Gibson has intimated the railway went into the hands of a receiver. We were able to deal with the receiver and get immediate relief.

MR. WIGGIN: I would like to ask Mr. Mabee what the thickness and pressures are in his system?

MR. MABEE: What the pressures are?

MR. WIGGIN: The general thickness of the pipes used and the pressures?

MR. MABEE: We are using Class B Pipe, American Water Works Association, Standard. The pressures fluctuate from 65 to 90 pounds, sometimes to 100. This Class B pipe, however, has been used only for the last seven years. Previous to that we used our own specification which was half way between B and C, but we found it was not necessary to have that much metal. We have been doing very well. There has been no increase in breaks since we put in the Class B pipe.

MR. WIGGIN: Have you used any of the centrifugal pipes?

MR. MABEE: No, we have not. We are sticking to pit cast.

MR. WIGGIN: Not any centrifugal?

MR. MABEE: We may come to it, but we will let the other fellow try them before we experiment with them.

CHAIRMAN GIBSON:[4] When this paper first came up I began to search our records at Charleston for breaks. I went back only ten years, because that was about the limit of time on which I had definite information as to breaks. We had from the records of the old water company and Proceedings of Council a pretty good historical record of the pipe from the period of the first installation of the works in 1880 to date. We have classified the breaks in Table 3 according to the years in which the pipe was laid, and the nature of the break.

As you can probably judge from my previous statements here today Charleston is a coastal city, having sandy soil, with a great deal of what we call pluff mud along the river and creek shores. This pluff mud is the ocean bottom ooze highly contaminated with vegetable matter and in many places this has been filled in forming a portion of the city area. Most of the city system, about fifteen miles or thereabouts, was laid in 1880 and is in the older portion of the city along the waterfront. While I am not going to read all of these results I will say that out of a total of 141 breaks that have occurred in the past ten years 53 occurred on pipe that was laid in 1880. The fifteen miles, a good bit of it, was 4- and 6-inch pipe and none of it above 10-inch.

We find that there is no record of any breaks in pipe laid between 1891 to 1899. The explanation of this is that there was very little pipe laid during this period about ten years. The company was privately owned and going through the vicissitudes of receivership, etc., and they had no money and no revenue and they did not lay any pipe. In recent years we have had a spasm of street paving due to the desire of the public to ride on rubber without any "thank you moms" and, of course, in the process of new paving they also put in new sewers and storm drains. Unfortunately we stepped into municipal ownership about at this transition period of street sewer and drainage improvement. It was a trying period and I must say with regret that the

[4] Manager and Engineer, Water Department, Charleston, S. C.

TABLE 3

Record of breaks in mains of Commissioners of Public Works, Charleston, South Carolina

LAID	NUMBER OF FEET OF PIPE IN SYSTEM AT THE END OF EACH YEAR																																						TOTALS
	20343	245588	38472	10170	20820	254461	39868	13701	268293	39868	10170	23206	286961	42399	10170	13782	23244	289788	25851	290368	25941	289697	47543	26165	303697	47546	18713	15993	25612	301602	47552	25612	307362	48820	307500 + or −				
	1922				1923				1924			1925					1926		1927		1928			1929					1930			1931			1932				
	4″	6″	8″	12″	4″	6″	8″	16″	6″	8″	10″	4″	6″	8″	10″	16″	4″	6″	4″	6″	4″	6″	8″	4″	6″	8″	10″	16″	4″	6″	8″	4″	6″	8″	6″				
1880		3	1		1	3	1		6	1	1	1	2		1		1	8		2	2	5			2	2	1			5	1		1	1	1					53
1881																					2											1								3
1884	1					1							1								1												1							5
1885													1							1		1																		3
1887																				1																				1
1889					1	2												2																1						6
1890	1					1			5				1					1							2				1	1		1	1		4				19
(No record of any breaks in pipe laid during years 1891 to 1889, Inclusive																																							
1900		1			1																			1																3
1902																																			1				1
1903													1																											1
1904																														3										3
1906																									1															1
1908																						1													1					2
1909																		1	1																					2
1912																		2		3		1	1		1	1					1									10
1913																														1										1
1917												1					1																							2
1918									1					1																										2
1919																				1																				1
1920						1														1									1											3
1922				1		2	1	1								1		2		1								2			1									12
1923																	1																							1
1924									1																					3										4
1925																									1															1
1928																															1									1
Totals....	2	4	1	1	3	10	2	1	13	1	1	2	6	1	1	1	3	16	1	10	5	8	1	1	7	3	1	2	2	13	4	2	4	1	7				141
	8				16				15			11					19		11		14			14					19			7			7				141

public works department did not seem to grasp the situation that the water department was now owned by the citizens. They paid very little attention to our piping system, built man holes, drove sheeting without any rhyme or reason, without any effort to determine where our mains were, with the result that we have had a good many breaks in the last five or ten years due to that method of construction.

Probably 80 percent of our breaks in the newer portion of the city, or the newly sewered portion are due to indifference in construction and backfilling of these sewers and drains causing settlement of our

TABLE 4

Ratio of breaks to pipe installed, 1 for each 100,000 feet of pipe, Charleston, South Carolina

YEAR	SIZE, INCHES					
	4	6	8	10	12	16
1922	9.80	1.60	2.60		9.80	
1923	14.50	3.90	5.00			7.70
1924		4.80	2.50	9.90		
1925	8.60	2.10	2.40	9.80		7.30
1926	12.90	5.50				
1927	3.90	3.40				
1928	1.90	2.70	2.10			
1929	3.80	2.30	6.30	5.30		12.60
1930	7.80	4.30	8.40			
1931	7.80	1.30	2.00			
1932		2.30				
Totals.....	71.00	34.20	31.30	25.00	9.80	27.60
Averages.....	7.90	3.10	3.90	8.30	9.80	9.20
Average for 11 years....	6.40	3.10	2.80	2.30	0.90	2.50

mains. In the older portion of the city the breaks may have been due to electrolysis and the action of the salt sea water in this ocean deposit of ooze and mud. I have tabulated these breaks into breaks per hundred thousand feet of pipe laid. I must congratulate Mr. Mabee and other members, who spoke to us, on the small number of breaks that they have had per mile of pipe. Just roughly, calling a hundred thousand feet twenty miles, our lowest break is about one break per hundred thousand feet or about 0.045 per mile (table 4).

REPORT OF COMMITTEE ON CONTROL OF TASTES AND ODORS

The Committee on Control of Tastes and Odors in Water Supplies was formed primarily to present at this meeting a report on the present status of this problem. Unfortunately, it was not formed and organized until quite late in the year and has, therefore, not had an opportunity of doing all that it has set out to do.

Control of tastes and odors is probably the outstanding problem of water purification men today. Within the last few years great strides have been made, but it is still evident that considerable work and knowledge are necessary before the water works man can be sure of consistently producing a water free of tastes and odors.

Originally this committee hoped to present to you a comprehensive report or statement on the proper use, limitations, etc. of each of the common methods of control, but it soon became evident that this object would be hard of attainment, at least in the short time available. The entire subject of tastes and odors is so much a matter of opinion that merely summarizing the reported experiences of plant operators fails to give more than a resume of the honest opinions of these workers, and brings home the fact that these honest opinions conflict in many cases. This condition exists largely because of the limitations of the present method of odor determination in Standard Methods, which relies too much upon the personal equation as to quality and intensity. Some progress has been made in developing methods that overcome these objections and appended to this report is a discussion of several of them. It is hoped that more study can be given to them.

A large percentage of taste and odor compaints is probably due to the more common causes such as improper control of chlorine dosage, algae growths, presence of hydrogen sulphide, the use of water from stagnant layers in basins and reservoirs, the presence of iron and iron bacteria, etc. The elimination of trouble from such causes lies in better control and use of proven measures.

Other complaints are not so easily disposed of. These often require special treatment and considerable research as to which avail-

able method or combination of methods may be successfully applied. The methods available are given below with a brief statement as to the tentative opinion of the committee as to their applicability. As has been stated, insufficient time has been available to give more than this outline, which represents largely the experience and opinion of the members of the committee, dealing, however, with a considerable number of installations.

AERATION

Aeration is and has been an effective means of odor elimination in a number of plants. It is effective in removing unpleasant gases such as hydrogen sulphide, those caused by some algae, and in improving water from stagnant areas. Aeration probably was more effective before the introduction of chlorine complicated the problem by the introduction of chloro-substitution products, the removal of which is not so easily accomplished. This and the fact that many aerators are very inefficient may be an explanation of the inadequacy of this method in many cases.

COPPER SULPHATE

Copper sulphate in itself is not an odor removing agency. Its use in controlling algae or other plankton forms in the source of supply is recognized as an important method of keeping tastes and odors from organisms within bounds.

AMMONIA-CHLORINE

The ammonia-chlorine treatment apparently finds its greatest field of application in the prevention of chlorophenol tastes and in enabling the carrying of higher residual chlorine values in distribution systems without taste or odor. It is useful in abating phenol tastes when such substances occur in the water supply in small amounts. Apparently there are limits to the usefulness of this treatment, although just what concentration of phenol cannot be successfully handled is hard to say definitely at this time.

In the control of algae the ammonia-chlorine treatment apparently is of little value, although reports by various operators offer conflicting testimony on this score. It is the opinion of this committee that probably the greatest benefit of the chlorine-ammonia treatment is in the prevention of chlorophenol tastes when the phenols are in considerable dilution, in securing continued sterilization throughout

the distribution system by maintaining residual chlorine values that without ammonia would almost surely produce taste, and in the prevention of chlorine substitution tastes in water free of taste before treatment with chlorine.

SUPER AND DE-CHLORINATION

This method is used, so far as known, on a plant scale only at Toronto and Glencoe, Illinois. Some other plants use this method at intervals. Its use has not been general enough to allow giving an opinion as to its general applicability.

PRE-CHLORINATION

Pre-chlorination is very much like super-chlorination except that dechlorination is not required. It has been reported as being in successful use, particularly in controlling algae growths and thus preventing tastes and odors. Hydrogen sulphide removal with chlorine in deep well systems pumping directly into the distribution system appears to be a simple cheap way of removing this objectionable gas if aeration cannot be installed.

POTASSIUM PERMANGANATE

This material has been successfully used in only a very few plants. Its use seems limited to the prevention of tastes due to phenols.

ACTIVATED CARBON

Probably one of the greatest aids to modern methods of taste and odor removal or prevention has been the rather recent adaptation of activated carbon to water purification practice. Used in granular form as a filtering media it has been studied in detail by Baylis, Norcom and others. One such plant installation has been made at Bay City, Michigan, and it is understood, at several other plants. There are also some plants of this type used in industrial plants preparing food products in which the water used must be free of taste and odor.

Powdered activated carbon is now the most generally used form of this material. Fed by various means into some part of the purification process ahead of filtration its use has been remarkably successful. At present its application is most generally made to the water leaving the settling basin, filtration allowing its removal. Applied ahead of settling, removal of odors have been accompanied in some instances by improvement in the character of the settling basin

sludge. Of the numerous installations and uses of this material, only a very small number have reported any taste or odor that cannot be removed, and it has been brought out that quite likely an increased carbon dose would have been successful. At the present time, activated carbon either in the granular or powdered forms seems to give the most promise of general successful applicability.

COAGULATION, SEDIMENTATION, FILTRATION

Coagulation, sedimentation and filtration are not generally considered taste and odor removal agencies. It should be pointed out, however, that they are of considerable importance.

ABSORBENT CLAYS, FULLERS EARTH, ETC.

These materials are being tried experimentally in several plants for removing tastes and odors.

APPENDIX A

METHODS OF DETERMINATION

Standard Methods of Water Analysis provide a method for determination and recording of odors in water supplies. This method has not met the entire needs of laboratory studies of taste and odor troubles because of several reasons:

(1) The attempt to classify the intensity as objectionableness of an odor by considering its effect on the consumer introduces an unknown factor hard to evaluate.

(2) The gradations of intensity are not uniform and therefore do not allow reproducible results. The first three steps are probably very close together quantitatively, the fourth is considerably further removed, and the fifth jumps to infinity.

(3) No provision is made to overcome the natural immunity laboratory workers and consumers acquire in continuously testing the same water.

Attempts have been made to overcome these objections by the addition of other descriptive epithets as for instance: o—oily; T—tarry; Sw—sewage; x—medicinal; v—varnish. Others have applied their own definition to the numbers describing intensity, as for instance having the entire range from no perceptible odor to absolute unfitness covered in five equal steps, rather than as in Standard Methods.

The danger in becoming accustomed to any particular odor through

working continuously with it presents a difficulty not easily overcome, except through an out of town observer's checks. Even with this precaution, it is often impossible to remember whether a particular odor was marked "5" or "3" or "4."

The determination of taste and odor involves the personal factor to such an extent that this must somehow be considered. Any test, however, that will aid in the finding of tastes and odors in a supply before the consumers become aware of it will be of benefit. Several such tests have been proposed and used with considerable success. These methods are given in the hope that they will be of benefit to other operators and be an incentive to further study along this line.

Method A

(As used by plants of the American Water Works and Electric Co.)

Standard Methods for hot and cold odors are followed.

In addition the following "taste" method has been used for the purpose of giving the plant operator a means of quickening his senses so that he will detect the presence of a taste or odor in the water before a consumer does.

The samples are concentrated by distillation. The concentrates are obtained by distilling at a rate of approximately 10 cc. per minute until a 100 cc. sample is obtained. In some cases concentrations of 2 to 1 are used for routine tests. In other cases higher concentrations such as 5 to 1 or 6 to 1 are used. The concentration tests are made to detect taste and odor producing substances, to identify them, and to determine to what extent they are being removed at various steps in the treatment. In addition to giving more of the substance to work with, the distillation makes it possible to make taste determinations upon raw and partially treated samples with safety. In the case of many observers the results for taste are much more reliable than tests for odor.

In making the record for taste, a number is used to express the intensity as in the case of odor, i.e. 1—very faint, 2—faint, etc. The observers are trained to recognize the taste and odor producing substances likely to be present. If found in troublesome amounts the condition is observed by two persons, so that the record of intensity will be more accurate.

Whatever concentration is selected as suitable is then followed as routine. The numerical value is placed upon the intensity observed

in the concentrates and so recorded. No attempt is made to change the result obtained upon a concentrate back into terms of intensity for the original sample. We use the same concentration day after day and base our opinion of the results of the treatment upon this record. In some cases it has been necessary to use concentrations such as 6:1 upon the effluent at the plant in order to be sure there remains no substance present which can later cause taste and odor troubles in the distribution system.

If experience shows the substances in the water are likely to cause taste upon the addition of chlorine in the plant treatment, a chlorine solution of known strength is added to give a residual of approximately that used in the plant to the samples before they are concentrated. In some cases it may be advisable to make the concentrations with and without the addition of chlorine to discover if the troublesome substances are present. A record of both the odor and taste is made upon the freshly prepared concentrates. The odor which comes over with the first part of the distillate can also be observed as an aid in identifying the substance.

Concentration of the sample by boiling may make some change in the character of the odor from that observed in the raw water. This may be due to a number of causes which are usually unimportant in so far as the tests are concerned and can be disregarded. As soon as the substance is identified in its "new" form the record can be made upon this basis. In fact, it has been observed that the taste and odor of the concentrate may resemble more closely that found in the distribution system than anything discovered as an odor in the raw water.

One of the most troublesome problems in taste and odor control comes from the disagreeable tastes and odors which develop in the distribution system. Some operators overlook the importance of this condition, taking the position that, if the plant effluent is free from taste and odor, treatment cannot be at fault.

The concentration tests make it possible to predict whether or not a taste or odor is likely to develop later. This, of course, is very important because the true conditions at the plant must be known first if a corrective treatment is to be worked out and applied at the proper time. It is assumed that the substances are varying in intensity in the raw water at the time, which is usually the case when the dilution is so great it is difficult to identify the source of the trouble.

The complex nature of the ordinary taste and odor producing substances seems to preclude the possibility of developing a direct means of measurement. The fact that they are present in such great dilution would make any accurate determination difficult.

These concentration tests have been used upon samples collected in surveys of streams in order to locate the source of the substance causing the trouble. By observing the taste in the concentrates of the river samples it is possible to locate the area from which the troublesome substance is coming. A survey of this area can then be made to locate the actual source of the pollution. It may be the substance cannot be identified by the taste imparted to the samples, but if the method leads to the source of the pollution, it has served the same purpose.

It should be added that a person with a delicate sense of taste will probably detect some kind of a taste in the distillate from any water. This may interfere with the first observations made in solving a taste and odor problem. However, the observer can soon learn to recognize those substances which are objectionable when present in his water supply.

Method B

This method developed by Chas. H. Spaulding consists briefly in finding the highest dilution of the unknown which can be differentiated from odor free water, making use of a scientific method of reporting odor values used in other fields.

Preparation of dilution water and flasks. A good quality of distilled water must be used for dilution purposes. This water is boiled until odor free and then cooled to room temperature. The boiling period required varies in length. Usually approximately 10 percent of the water volume is evaporated.

In making the determinations use 500 cc. Erlenmeyer flasks covered with watch glasses or small inverted beakers. The flasks must be clean and the last traces of odor removed by boiling distilled water in them for a period of five minutes. This water is discarded and the flasks allowed to cool.

Procedure. To determine the approximate odor put 100 cc. of the odor free water in each of two prepared flasks marked A and B. If hot odor is to be determined warm the water in the flasks to about 70°C. Add to Flask B a portion of the sample, shake both flasks vigorously, uncover, and determine whether or not the water in

Flask B has more odor than the distilled water in Flask A. If odor is present repeat using a smaller portion of the sample and if odor is absent add more of the sample to Flask B until an odor is present. More reliable results will be obtained if the flasks are interchanged so that the analyst does not know the flask in which he may expect to find odor.

After the approximate odor is obtained the odor may be determined accurately by setting up a series of flasks containing various portions of the sample. By running up the series the dilution is found which gives the first perceptible odor. If more than 5 cc. of the sample are required the quantity of the dilution water is reduced so that the final volume is 100 cc.

Expression of results. The numerical value for the odor is the reciprocal of the threshold dilution expressed in hundredths. For example, if 4 cc. of the sample on dilution does not give an odor while 5 cc. does, the numerical value of the odor is 11/5 or 20. A maximum odor of 1000 is obtained when 0.1 cc. of the sample gives a perceptible odor on dilution.

Remarks. Care must be taken in making this determination that there are no odors close to the nose. The hands must be free from the odor of tobacco, soap, etc. Odors from wooden bench tops may also interfere.

Method C

This method may be classed as a quick modification of Method A. This consists in placing 100 cc. of the sample in a distilling flask and distilling off about 5 cc. into a small Erlenmeyer flask. The distillate is then warmed and agitated and the odor observed. The same descriptive epithets and numerical values are employed as in the regular Standard Methods.

APPENDIX B

THE DETERMINATION OF PHENOL IN WATER

Probably no other offending material in the cause of tastes and odors has received as much publicity as phenol. A discussion of the methods of determining this substance is given in the hope that more data will become available as to the concentrations encountered in water supplies, and what concentrations can be successfully treated.

There appears to be general indecision among the laboratories

confronted with the problem of making quantitative determinations of phenol in water supplies, regarding the method of analysis to be used. The Ohio River and Great Lakes Board of Engineers which represent the health departments of the states of Pennsylvania, Ohio West Virginia, Virginia, Tennessee, Maryland, North Carolina, Kentucky, New York, Michigan, Indiana, Illinois, Wisconsin and Minnesota, created a committee to study the various available tests and to propose a uniform technic to be used by its member laboratories.

After some study the committee, which is comprised of Messrs. F. E. Daniels, O. O. Malleis, C. O. Pierce, R. D. Scott, F. W. Sperr, Jr., L. M. Wachter and W. H. Fulweiler, Chairman, reported on November 3, 1929, that they had not up to that time been able to decide on a method for quantitative determination of phenol in coke oven plant effluents. Their work indicated that the Skirrow method seemed to be best fitted as a referee method and the Koppers method the most promising as a control method. No mention was made regarding any methods for quantitative determination of minute quantities of phenols in water supplies.

The committee, however, proposed a provisional method for testing for phenolic taste and odor after chlorination. The test is described . . . . "to measure the so-called phenolic tastes and odors in water after chlorination and to determine whether or not the taste and odor is due to the presence of substances volatile and steam from acid solutions, e.g., phenol-bearing wastes." The test which is qualitative and crudely quantitative has two separate and distinct objects:

(1) to be used by plant operators to determine whether chlorination will produce taste and odor in a water supply and whether such tastes are due to the presence of phenolic wastes.

(2) to be used by operators of plants discharging effluents to determine what dilution is necessary to free such effluents from the possibility of producing taste and odor in a water supply after chlorination.

R. D. Scott, Chief Chemist, of the Ohio State Health Laboratories, employs the Gibbs method as modified by Baylis, and later by Theriault, for determination of phenols in water. He states, "In my opinion the method is reasonably accurate and satisfactory and is the best available at the present time for the determination of minute

quantities of phenol." For the determination of phenols in coke oven plant wastes he used a modified Skirrow procedure for preliminary purification, followed by a bromine titration.

On the other hand, L. M. Wachter, Chief Chemist of the Division of Laboratories and Research of the New York State Department of Health, in a private communication states—"Up to the present time, we have felt that the Fox and Gage method, outside of the fact that it has certain limitations, is the most satisfactory in our hands." Mr. Wachter did considerable work with the modified Gibbs method. He found that with greater concentration of phenols the results were inaccurate unless the distillates were diluted before the phenol reagent was added. He suggests "that the description of the method should contain details pointing out definitely the limitations as to the quantities of phenol which may be contained in a particular tube to which phenol reagent is added. In other words, it would seem that when dealing with samples containing relatively large amounts of phenol, dilutions should be made in the tubes to which reagent is added so that their phenol content would be within the range where the reagent would give approximately accurate results."

In Chicago, the Baylis modification of the Gibbs method has been used since 1927 by Physical Chemist John R. Baylis at the Experimental Filtration laboratory, and by Chief Chemist Wm. Marti of the Health Department Laboratories, with very satisfactory results. The Chicago experience has been that the modified Gibbs method will give fairly accurate results with concentration of phenol as low as 5 parts per billion and that with suitable concentration, amounts as low as 1 part per billion could be determined. The method also appears to be specific for phenolic compounds and is not affected by interfering compounds as are some of the other methods.

Baylis reports that the result of his work fail to confirm Wachter's observation that there are inaccuracies in the method when the chloroimide reagent is added to distillates having high phenol content. Baylis believes, however, that when the phenol content is over 500 parts per billion the color developed is too intense to be compared accurately and it would be advisable to dilute the original sample or the distillates sufficiently to have the color within the range where it can be compared without much error with the standards. For samples containing more than 100 (?) parts per million phenol a method such as the bromine titration should be used.

It is understood that the Committee on Standard Methods of

the American Public Health Association has had the Gibbs method under consideration for some time for inclusion in the Standard Methods. The method under consideration is essentially that originally proposed by Baylis, with slight modifications.

The Chicago Health Department laboratories have found that considerable error in the phenol determination, due to the reduction of phenol content by bio-chemical oxidation during the interval between collection and analysis of sample, could be avoided by adjustment of the hydrogen ion concentration of the sample immediately after collection. It is understood that these findings will be published in the near future.

SUGGESTED RECOMMENDATIONS

1. The Ohio River and Great Lakes Boards provisional method for test for phenolic tastes and odors after chlorination, to be used for routine plant control where qualitative or crudely quantitative results are all that are required.
2. The modified Gibbs method to be used where accurate quantitative determinations of phenols in water supplies are required.
3. For determination of phenol in coke oven plant effluents, the procedure used by Scott, is recommended.

MARTIN E. FLENTJE,[1]
Chairman.

DISCUSSION

H. H. GERSTEIN:[2] Reliable information regarding the limitations of the various taste and odor elimination treatments should be of incalculable value to the water works profession.

In general, it has been a matter of "cut and try" to determine which of the treatments was the most effective for the elimination of a particular taste condition.

Treatments have been reported unsuccessful, where, as a matter of fact, a larger dosage of chemical or a longer period of contact would have produced good results. On the other hand, treatments which have been used with success on certain tastes have been found ineffective for tastes of a similar character but of more pronounced intensity.

[1] Community Water Service Company, Harrisburg, Pa.
[2] Sanitary Engineer, Department of Public Works, Chicago, Ill.

As stated by Flentje, the lack of an adequate measuring stick for evaluating intensity of taste and odor has been a serious handicap in making accurate comparisons of results obtained from different plants. I am sure that no one will dispute the statement that the method for determination of taste and odor in Standard Methods is totally inadequate and inaccurate.

An illustration of the inaccuracy of the method is shown by a recent experience with two waters each being rated as having an intensity of *5*, which according to Standard Methods represents taste intensity of a water which is practically undrinkable. It was determined that intensity of taste of one sample was actually about 10 times that of the other sample, yet because of the fact that both samples were practically undrinkable there was no alternative, but to give each the same intensity value of 5, in spite of the fact that a much more intensive treatment would be required to eliminate the stronger taste.

The adoption of the Spaulding method for determination of the taste threshold point—the maximum dilution with tasteless water which will give taste—would be a step in the right direction. When properly used it should be a great aid in making accurate comparison of the effectiveness of the various taste and odor elimination treatments.

CROSS CONNECTIONS WITH PUBLIC WATER SUPPLIES

By S. B. Morris[1]

In this paper I shall present and discuss only problems relating to cross connections due to faulty designed or installed plumbing fixtures or other devices installed by consumers and generally connected to the public water system.

As this subject was very ably presented to this Association by Mr. Joel B. Connolly[2] at the St. Louis Convention two years ago, I shall not attempt to fully cover all phases of this very important subject. I shall presume that each of you is already familiar with the general facts which were brought to your attention by Mr. Connolly and the discussions of his paper.

The importance of the subject, however, justifies some brief review of the typical cross-connection which may exist in plumbing fixtures. It is not sufficient that fixtures should be so designed and installed as not to permit flow of contaminated water by gravity into fresh water piping. Experience demonstrates that there are times when high partial vacuums occur on the water mains or within buildings. Therefore, plumbing fixtures are at times subjected to such partial vacuums. When valves are opened such fixtures must not permit siphonage. The limit of siphonage is of course about 33 feet.

It is obvious that any fixture having a submerged inlet is liable to siphonage when partial vacuums occur on the water supply piping. Many fixtures such as flushometer valves are operated by differences in pressures on hydraulic plungers or pistons. As flushometer valves are generally closed by the accumulation of water pressure acting against a plunger, such valves are opened and remain open when a partial vacuum occurs, thus permitting siphonage to occur.

It is, however, but a few years since this subject first received the serious attention of engineers in the United States. New types of plumbing fixtures and other equipment to be connected to consumers' water piping are constantly being developed and placed on the market.

[1] Chief Engineer and General Manager, Water Department, Pasadena, Calif.

[2] Journal, April, 1931, page 495.

Cross connections in plumbing have been defined by the American Society of Sanitary Engineering as any construction or system whereby it is possible and/or probable, that a potable water supply may be contaminated. It is essential to consider, therefore, not only the proper construction of the individual fixtures or appliances, but also their usage in connection with each other, and their manner of installation.

FIXTURES INVITING CROSS CONNECTIONS

In general, offending fixtures are: Plumbing fixtures with under rim water supplies, such as water closet bowls, urinals, slop sinks, bidets, lavatories, bath tubs; tanks and storages of all kinds where water inlet is below the water surface or through stoppage the inlet may be flooded; direct flushing valves, undoubtedly the greatest hazard; hospital fixtures such as sterilizers, slop sinks with bed pan cleansing jets, and various hydrotherapeutic equipment; water softeners, air conditioning devices, dish washing machines, douche cabinets, ejectors, swimming pools, fountains, filters, air conditioning equipment and other devices which may permit a cross connection between the water supply piping and contaminated water in the fixture or the drainage system to which the fixture may be connected. Danger of contamination from such fixtures is generally incident to a temporary fall of water pressure on the house service itself or on the water distribution system which may permit a partial vacuum to be formed.

ACTUAL EXAMPLES OF CONTAMINATION FROM FAULTY FIXTURES

Some examples of such contamination which have come to my attention are:

In the residence of a physician in a neighboring city there was complaint of highly contaminated and offensive water on occasion. Investigation showed that the small sized piping and limiting water pressure caused the contents of the second floor siphon jet toilet with flush valve to siphon into the water system whenever a toilet was flushed on the first floor.

In another instance water showing direct sewage contamination was drawn into the kitchen sink from the back siphonage of a toilet on a higher floor.

A plumber called at a residence to open up an overflowing toilet on the second floor. He turned off the water at the curb and opened a

valve downstairs to drain the water piping. Much to his consternation the highly offensive contents of the flooded toilet bowl were drained out through the water system.

EXAMPLES OF CONTAMINATION FROM FAULTY PIPING

Then there are many cases of faulty piping arrangements. Investigation of a complaint from a soda fountain revealed contamination through wrong valves having been opened permitting water to be taken from the circulation line of a refrigerator operating with a cooling tower. The fault here was that make-up water was drawn directly into the circulation piping instead of over the rim of the cooling tower storage.

Many thousands of gallons of lime water were discharged into the distribution system from a lime water softener plant. Again, this was due to faulty operation of valves and failure of check valves. In all such cases water should be discharged over the rim of the storage tank only.

In another instance complaint of excessive chlorine which tarnished the silverware in a school cafeteria was found to be due to the kitchen tap having been made on a swimming pool circulation line instead of on the potable water supply system. Fortunately continued presence of free chlorine in the swimming pool water prevented any outbreak of disease as well as tarnishing the silverware and affording means by which the contamination was discovered.

EXAMPLES OF CONTAMINATION THROUGH CARELESS OPERATION

Some instances when contamination has occurred are due to the carelessness of individuals, which cases are entirely outside the control of ordinary public authority. Showers and drinking fountains at a municipal swimming pool were very grossly contaminated to the extent of most offensive odor of raw sewage by the carelessness of a sewage plant operator. A centrifugal sewage pump lost its prime so frequently that the operator finally wired a garden hose from the potable water supply directly to a blow off cock on top of the discharge shell of the sewage pump. The pressure on the sewage pump was about 60 pounds per square inch compared to about 50 pounds on the public water supply system causing raw sewage to flow into the water system.

Investigation of a large amount of air in the water distribution system revealed the fact that an automobile machine shop and garage

were regularly pumping air back into the water system to make the water meter run backwards. This was finally discovered when the meter gave a negative registration.

Such examples as the latter, while not subject to ordinary prevention by regulation or inspection are fortunately very rare. In spite of so many crimes committed against the public, I do not know of any cases where the water supply has been intentionally contaminated, although it could be done with little chance of detection.

TESTS OF PLUMBING FIXTURES UNDER PARTIAL VACUUM

In order to determine more accurately the effect of a partial vacuum upon various plumbing fixtures and devices, it was decided to test such fixtures and devices under a controlled vacuum and then to correlate the results with actual field conditions by testing the amount and duration of vacuums occurring on the distribution system.

Walter Putnam, Superintendent of Building and J. B. Davis, Plumbing Inspector for the City of Pasadena, tested a large number of toilet bowls and various flushometer valves. In brief, these tests demonstrated the fact that all types of flushometer valves tested showed opening of the valve under partial vacuum and the complete inadequacy of any by-pass venting to break such vacuum. The only flushometer valves which did not open under partial vacuum so as to effect siphonage were those installed with check valves on the supply lines. But dependence cannot be made on a check valve. In some instances, we have learned of check valves, furnished by the manufacturer integral with his fixture, having been removed by plumbers to overcome objectionable noise.

A glass tailpiece was installed between the flushometer valve and the toilet bowl so that observation could be had of the behavior of the valve and bowl under partial vacuum. It was found that all siphon jet bowls completely emptied their contents up through the glass tailpiece and through the flushometer valve into the water system.

It made little difference whether a side outlet or top outlet bowl was used. Nor did it make any difference whether the flushometer valve was six inches or any other reasonable distance above the bowl.

All bowls with siphon jet emptied their contents down to the elevation of the jet in 30 to 40 seconds when a relatively high vacuum of 15 to 18 inches was applied. When the vacuum was maintained at

only 2½ inches the contents of the bowl were siphoned out in less than 5 minutes.

In the case of wash-down type bowls without the siphon jet any contamination adjoining the under rim water outlets is drawn into the water supply piping by the vacuum, or if the bowl is flooded through stoppage then the contents of the bowl above the under rim outlets is siphoned into the water system.

Inspection through the glass tailpiece demonstrates the effect of the partial vacuum. Under high vacuum there is a substantial spray of upward moving water mixed with air. When only a low vacuum such as 2½ inches is applied there is only a mist with a film of upward moving water clinging to the inner sides of the glass.

U. S. BUREAU OF STANDARDS RECOMMENDATIONS

The United States Bureau of Standards published in 1929 a report on "Recommended Minimum Requirements for Plumbing," at Page 28 this report provides:

> "No water-closet or urinal bowl shall be supplied directly from a water supply system through a flushometer or other valve unless such valve is set above the water-closet or urinal in a manner such as to prevent any possibility of polluting the water supply.
>
> "No plumbing fixture, device, or construction shall be installed which will provide a cross-connection between a distribution system of water for drinking and domestic purposes and a drainage system, soil or waste pipe so as to permit or make possible the backflow of sewage or waste into the water supply system."

From the tests made it does not appear possible to affect the siphonage at all by the location of the flushometer valve.

FIELD TESTS OF EXTENT OF VACUUMS IN DISTRIBUTION SYSTEMS

The next step was to correlate these findings with actual field tests of vaccums occurring in service when water mains are shut down.

Topographically Pasadena is situated on a sloping plane with an average gradient of 1 to 5 percent, and in many areas there are steep hills. The elevation of service varies by as much as 1000 feet. Therefore, whenever water is shut off it tends to drain down to the lowest point and to create partial vacuums in the upper portion of the main or mains shut off.

In a number of tests where single mains were shut off and drained a block or two below the higher point of shut off, substantial vacuums

were developed for quite prolonged periods of time. These were all short runs of main 410 to 1175 feet in length and differences in elevation of only 10 to 20 feet. Mains were 4 to 16 inches in diameter. Partial vacuums quickly developed up to 57 to 88 percent of the full

TABLE 1

Results of typical tests for partial vacuum occurring in distribution system when water mains were shut off and drained

	TEST 1	TEST 2	TEST 3	TEST 4
Date, 1932	1/9	3/21	3/29	4/1
Size of Main, inches	4	16	4	6
Length of Main, feet	540	410	1175	863
Difference in elevation of gage and outlet, feet	10.5	12	18	20
Difference in elevation, inches of mercury	9.3	10.6	15.9	17.7
Vacuum in inches, following shut off, in minutes:				
0	7	0	0	0
2.5	5	2	2	3
5	3	3	2	12
7.5	2	4	2	8
10	1	5	14	2
12.5		5	10	0
15		6	4	
17.5		6	3	
20		6	2	
25		6	2	
50		6		
60		5		
Maximum Vacuum, inches	7	6	14	12
Vacuum in percent of difference in elevation, gage to outlet	75	57	88	68

Note: In test No. 4 the partial vacuum was 2 inches at 2 minutes falling to 0 at 2½ minutes when open garden valve was closed and increasing again to 3 inches at 2½ minutes. At 7 minutes with vacuum at 11 inches the garden valve was again opened to completion of the test.

difference in elevation as shown in table 1. In fact on a 16-inch main a 5-inch vacuum remained one hour after the water was shut off.

The danger of back siphonage under these conditions is very great. The vacuums are sufficiently high and the duration of time much longer than required to effect complete siphonage of any siphon jet bowl.

Not only do these tests demonstrate the need for regulation and supervision over plumbing fixtures and devices, but it also demonstrates the necessity of limiting the number and extent of shut-downs in the distribution system. Use of large sized tapping machines and other equipment to eliminate shutting off of mains is therefore justified for sanitary reasons in addition to economy and better service to the consumer.

These tests also demonstrate the value of opening fire plugs or other large vents near the high point when a shut-off is made so as to admit sufficient air to replace the water as the pipe drains, thus preventing material vacuums from forming in the mains. However, this does not prevent high vacuums from forming within buildings at elevations above the street.

TEST OF MOMENTARY VACUUM IN OFFICE BUILDING

Recording pressure vacuum charts taken from a large public building where the top floor pressure varies from 10 to 15 pounds per square inch and where pressures sometimes fall to as low as 5 pounds show that pressures for brief periods throughout the day fall below zero and substantial momentary vacuums are created. There are, no doubt, in most water systems high buildings where the static pressures are low and occasionally or even frequent partial vacuums may be formed either due to use of water on the lower floors or a general drop in pressure on a portion of the distribution system.

PASADENA ESTABLISHES PLUMBING FIXTURE BOARD

Proper inspection and control of piping and plumbing fixtures installed by the consumer on his own premises is generally entirely outside the jurisdiction or supervision of the Water Company or Department whose information and control generally ceases at the water meter. It is therefore of the greatest importance that proper cooperation be established with the Health and Building or Plumbing Inspection Departments.

In Pasadena we have not only long been coöperating in such inspections, investigations, and studies, but we have recently secured the adoption of an ordinance providing for a Plumbing Fixture Board. This board is composed of the Superintendent of Building, the Health Officer, and the Chief Enginer of the Water Department. The Board to be known as the Plumbing Fixture Board will have jurisdiction over all plumbing fixtures to be installed in Pasadena. It will have

the power to make tests of such fixtures or devices, to approve or disapprove such equipment, or the Board may lay down restrictions as to the manner of installation or the use of such fixtures.

A laboratory where ample vacuum and water pressures will be available under proper control is in course of construction. We believe this Board will not only be of much direct value in controlling the problem of cross connections in Pasadena, but will afford a very beneficial tie between the three branches of the City Government all interested in public health and sanitation: the Health Department, the Superintendent of Building and the Plumbing Inspector, and the Water Department.

DISCUSSION

ARTHUR E. GORMAN:[3] The feature of Mr. Morris' valuable paper on cross connection work at Pasadena which is most impressive in my opinion is the unusually long periods of partial vacuum reported in the mains. The length of time a potential hazard exists is an important function of its real danger—a point which is very often overlooked in academic discussion of cross connections and their public health significance. Many persons, especially manufacturers of plumbing equipment, are willing to concede the health hazard of flushometer connections, but frequently dismiss the *degree of danger* by a statement that the probability of contamination is remote because the partial vacuum is not likely to be of long duration. Mr. Morris' finding certainly weighs heavily against this line of reasoning and contains valuable quantitative data, the sort of information more cities need to meet their cross connection problem properly.

Mr. Morris states that the check valves as a protection against back siphonage from toilet bowls are not to be relied upon. No doubt many water works men agree with him. Recently I had occasion to discuss this subject with Mr. Joel I. Connolly, Chief of Bureau of Sanitary Engineering, Chicago Board of Health, who—you will remember—made the important contribution on the cross connection problem at our St. Louis meeting. He told me that his experience with check valves on toilet bowl connections has been otherwise and that the degree of protection given is very substantial. At least it is an additional safeguard of real merit and where the expense of changes—especially on old equipment—is an item, the check valve is a compromise not out of line with the health risks in-

[3] Engineer of Water Purification, Bureau of Engineering, Chicago, Ill.

volved. A simple solution of the flush valve problem would seem to be a non-mechanical air vent, which I understand is being given careful attention by manufacturers.

To those interested in a detailed technical discussion of this toilet bowl cross connection problem I would refer them to a most valuable paper by M. E. Link, Director of the Stuttgart, Germany, Water Works, appearing in the May 25, 1929 issue of "Das Gas und Wasserfach" No. 21. This is a very important paper.

The Pasadena coöperative board for studying the merits of this type of cross connection is a sound effort toward a rational solution of this important public health and water supply problem. More such boards are needed.

I have a strong feeling that the most lasting and in the final analysis most direct way to overcome these health hazards in plumbing is to take our data to the manufacturers and sell them on the merits of our stand. They need the viewpoint of the health and water works official and we need their coöperation and not their antagonism.

The period of economic depression which our country is now experiencing has much significance as regards cross connections in industrial plants. Many large plants are operating on a part time basis, others have been shut down temporarily, and still others have been abandoned entirely. Shut downs are always periods of change. New plants starting up where old ones of a different character existed will require remodelling. New operating officials will be in charge of plants, possibly with inadequate knowledge of piping connections. The day of important piping changes in plants is here. Water works and health officials will do well to re-check all industrial plants within their jurisdiction now. In Chicago we are finding this to be true. It probably is so in other cities. I fear that with a revival of industrial activity we are due for more outbreaks because of cross connections. In this regard let us not forget that more cases of typhoid fever were traceable in the decade 1920–1930 to unprotected cross connections than to any other cause.

After many years of experience with cross connections of all types I am frankly of the opinion that no water system is entirely free of them or ever will be. Their existence may be numbered in dozens and not in individual numbers as many report. The problem of the health and water works officials is to evaluate their potential hazards and require elimination and control accordingly. In many places we require extensive changes and inconveniences out of proportion

to the real hazards existing. This practice is bad and should be avoided.

The cross connection problem is entering a new era. First we had the downright hazard of large cross connections between contaminated water systems and public water systems. Then came the period of many minor across connections and such as swimming pools roof tanks, toilet fixtures, etc.—all potential dangers and requiring proper control. These two periods dealt with cross connections jeopardizing the public health through infection of public water supplies.

I believe we must now look for cross connections involving public safety against chemical poisons. In large cities where industries use such dangerous chemicals as arsenic, cyanide and acids extreme care must be taken that vats holding these chemicals are not cross connected with the city water system. Frequently these vats are provided with over the top connections to the city water system as a safety precaution, but workmen attach a short length of hose to the connection for convenience. When this is done and the end of the hose is submerged the poisonous contents of the vat might under certain conditions be siphoned back into the city water system.

This new uses of water in electric refrigeration will bear careful investigation. Recently in Chicago sulphurous acid was forced back into the water system of a building from an electric refrigerator. Ammonia refrigeration plants when cross connected with a city water system may cause serious corrosion to brass valves and fittings if the ammonia is forced back into the city water system. In Chicago complaints of blue water have been received, due to just such corrosive action.

In conclusion, I wish to compliment Mr. Morris for his very valuable paper and to caution all water works and health officials that never has the cross connection problem needed more serious attention on their part than right now.

J. B. WINDER:[4] At Dallas, we have spent considerable time and money in eliminating cross connections as outlined by the United States Public Health Service in conjunction with the State Board of Health. We also appreciate the importance of taking the necessary precautions as set forth in Mr. Morris' paper in regard to plumbing fixtures and devices.

[4] Superintendent of Water Works, Dallas, Tex.

I have in mind three different places in our State which were forced to make changes in their piping because of the siphoning of waste from the plumbing fixtures into the potable water supply:

1. In one city, a surgeon asked the nurse to bring him the instruments out of a sterilizer; upon looking in the apparatus, it was found that the water had disappeared. This sterilizer was of the under-

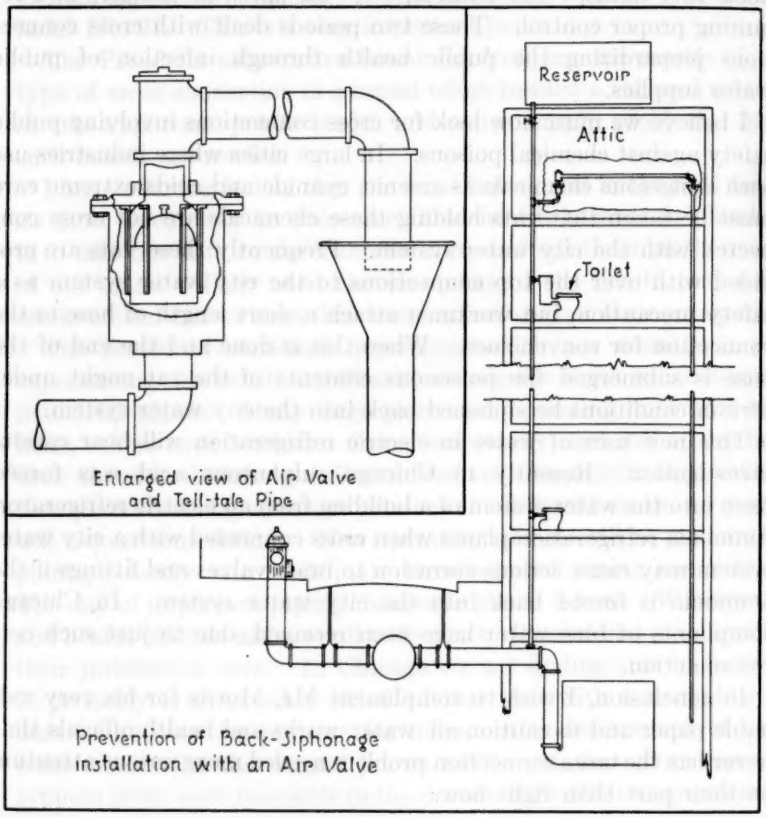

FIG. 1

feed type; the Water Department forces had drained the water main connected to the building, and the water from the sterilizer had drained into the main The remedy was obvious,—the supply line was changed to an overhead-feed.

2. Faulty flush valves to the toilet bowls gave trouble by siphoning out the bowls at a hotel in a small city. This trouble was corrected

by placing a vacuum breaker on each of the riser pipes reaching the roof. The vacuum breaker or valve used was a hard rubber ball, "enclosed in a bronze cage;" this ball would float and seat from the buoyancy of the water (fig. 1). Any lowering of the water on the negative side would automatically drop the ball, thereby opening the valve. So far, this valve has not failed to work. The hotel has been in operation now nearly two years. We do not recommend this arrangement as a "cure-all." However, it has some advantages over the old check valve used on commodes.

The third type of connection is where there were two different supplies of water brought together to supply a single fixture, such as a washstand or washer. This was remedied by separating the physical connections and allowing the supply bibbs to empty into a funnel-type fitting, in order that there be no physical connection.

I believe some action should be taken toward coöperating with the manufacturers of plumbing supplies in order that the defects may be eliminated. This procedure would perhaps be more effective than to attempt individual legislation through 48 state legislatures, and perhaps be conducive to greater uniformity.

IRON AND LIME IN REMOVAL OF MANGANESE

By E. C. Craig,[1] E. L. Bean[2] and R. W. Sawyer[3]

Red water or "dirty water" drawn by consumers from various sections of water distribution systems is still a common occurrence in many water works. Alkali treatment to reduce red water has often been followed by "black water" troubles, resulting from the precipitation of manganese which remained in solution before such treatment was used. These conditions are not confined to raw water supplies of variable quality, but are often encountered in the distribution of very clear raw waters and highly polished filter effluents that contain what has often been considered a minimum of residual impurities.

Water for the City of Providence, R. I. was pumped from the Pawtuxet River from 1870 to 1926, no storage being provided except in mill ponds along the river above the pumping station. The water was used without treatment of any kind for over 30 years until increasing pollution from mills and numerous rapidly growing mill villages led to the adoption of slow sand filtration in 1905. By 1917 the growing demand on the plant necessitated increasing the effective size of the sand in the filters and chlorination was adopted to supplement filtration at this time and thereafter continued in conjunction with slow sand filtration. In 1920 lime treatment was commenced, sufficient lime being used to neutralize nearly all of the free carbon dioxide. This addition to the treatment was made in an effort to reduce corrosion in the distribution system and particularly to stop the solvent action of the water on lead pipe. Analyses showed that this modification was successful in reducing the average lead content of tap samples from 0.29 to 0.05 p.p.m.[4]

In 1926 this supply, then using the highly polluted Pawtuxet River Water, was replaced by a gravity supply from a 37,000 million gallon impounding reservoir, called the Scituate Reservoir and controlling

[1] Superintendent, Scituate Reservoir Division, Providence, R. I.

[2] Chemist, Walter Purification Works, Providence, R. I.

[3] Assistant Engineer with Malcolm Pirnie, Consulting Engineer, New York, N. Y.

[4] J. N. E. W. W. A., 38, 1924, pp. 246–7, J. W. Bugbee.

about 93 square miles of watershed relatively free from pollution, with modern purification treatment consisting of rapid sand filtration with chemical coagulation and final pH correction. Alum coagulation, followed by settling, with lime applied subsequent to filtration, supplemented by chlorination when necessary, was very successfully used for five years to treat the soft, moderately colored water drawn from the reservoir.[5]

While each type of treatment from the original pumping plant to the present rapid filters seemed impeccable when put into operation, new developments in the art of water purification and the demand for water of a better quality in constantly increasing quantity demonstrated short-comings in the successive methods of treatment. The latest change was made in August, 1931, when coagulation with ferrous sulphate (copperas) and lime was adopted, placing all chemical treatment ahead of sedimentation and filtration in order to remove manganese and reduce red water troubles. Before describing the copperas-lime process and the results obtained from its use, it will be best, perhaps, to review briefly the sequence of events and conditions that finally led to the change in chemical treatment.

Although the treatment, as practised until the fall of 1931, supplied a filtered water having a color of 2, soap hardness of 18, iron of 0.03. manganese 0.18 p.p.m. and pH of 9.1, Providence was not immune from serious, although scattered, dirty water troubles. The situation that finally resulted in the change to the use of iron and lime treatment, became first apparent in October, 1929, when complaints of red water became frequent, reaching such proportions in November that a detailed investigation was initiated to discover the cause. Conditions were found to be worst in the dead-ends, which are nearly all of 6-inch pipe, and in those sections of the distribution system where the flow is slight. Numerous samples taken at and near the locations of complaints showed colors ranging from 30 to 120 p.p.m., with iron contents varying from several tenths to 6.0 p.p.m. During the latter part of December bleeders were installed on several of the dead-ends which were causing the greatest amount of trouble in the hope that conditions could be improved immediately by maintaining a steady flow in these pipes. The lime dosage was increased at the same time from 0.15 to 0.30 g.p.g. to raise the pH of the filtered water from 8.0 to approximately 9.0, which was continued for the next 20

[5] J. N. E. W. W. A., 45: 4, December, 1931, pp. 334–50, E. L. Bean.

months, with the exception of May and June, 1930. On some of the bleeders flows as high as 2,000 gallons per day were required to keep the color reduced to a point agreeable to the consumers. It was found generally that complaints do not become numerous until the color exceeds 25 p.p.m., corresponding in this case to an iron content of 0.4 p.p.m.

Operation was continued at this increased pH until May, 1930, when there was a distinct improvement in dead-end conditions, samples showing greatly decreased colors in most instances and a marked reduction in iron content. In order to ascertain whether the decrease in corrosion was due to bleeding or the high pH being carried, the lime dosage was reduced during May and June so that the filtered water again had a pH of 8.0. Although the bleeders were still in operation colors began to increase so that the pH was raised to 9.0 in July and has been maintained at or above that value to the present time. After raising the pH the second time it again took about six months before there was a definite reduction in the amount of iron being dissolved from the pipes in the distribution system and while no samples of pipe have been removed for examination it seems probable that the beneficial results of the increased lime dosage are due in some measure to the formation of protective carbonate coatings on the pipe. When excellent water was finally obtained on dead-ends, the bleeders were shut off to determine if the decrease in corrosive action was sufficient to maintain satisfactory conditions in those dead-ends which have been subject to bleeding. It was found impossible to obtain satisfactory water without bleeding.

The solution of lead from service pipes in the system, while it reached to 0.29 p.p.m. in the early days of the slow sand filters, has not been of sufficient magnitude in Providence for some years to cause any concern, but it seems worthy of mention that services which had been showing 0.02 p.p.m. of lead dropped to zero at the same time the solution of iron was shown to be decreasing in the dead-ends at a pH of 9.0. Since February, 1931, no analysis has shown the presence of lead when testing for one-hundredth of a part per million, indicating that the solution of this metal can be arrested by lime treatment of a soft, colored water, although some writers place the minimum solubility at a pH of 7.6 to 8.0 and express doubt as to the success of lime treatment in preventing the solution of lead in a water of this nature.

MANGANESE TROUBLES

Accompanying the red water complaints in the fall of 1929 were also a number on account of "dirty water" or "black sediment" in locations where unusual flows, such as are caused by fires, had occurred and in the areas serviced by high pressure which were being changed over at that time for operation on the low pressure system (operating at a higher pressure since the Scituate Supply has been in use), with resulting reversals of flow in some of the pipes. These "black sediments" were the first evidence of the presence of manganese in the distribution system, and their previous absence indicates that the amount of manganese precipitated must have been small prior to this time. It seems probable that lime dosage after filtration and bacterial action in the slime coatings in the pipes caused some of the manganese to precipitate in the distribution system at a pH of 8.0; coagulation and filtration having taken place at approximately pH 6.2 which is not high enough for the manganese in the raw water to be oxidized to those states in which it can be precipitated or adsorbed by the floc in the settling basins or on the sand in the filters.

Analyses of sediments in the distribution system during 1930 and 1931 showed increasingly large percentages of manganese and analyses of samples of the plant effluent and of tap samples in the City indicated precipitation of manganese at rates varying from one-half ton up to eight tons per year, as manganic oxide (Mn_2O_3), and of a total of ten tons present in the effluent during the year 1930–31 not less than half was precipitated in the distribution system. The first indications of manganese were discovered while a pH of 8 was being carried in the effluent and precipitation increased, as might be expected, when the pH was raised to 9 to remedy the red water troubles previously mentioned. At the same time that complaints from red water were waning, difficulties caused by the precipitated oxides of manganese began to increase so that complaints were numerous in the spring and summer of 1931. Laundries, particularly, suffered from the staining of clothes until their appeals for better water became almost frantic, which is not surprising in view of the fact that some samples of sediment from "black water" showed manganese as high as 80 percent, as manganic oxide.

Efforts to determine definitely the source of manganese in the raw water have not met with any great success. The manganese content of waters in tributary streams has been almost negligible as found by

analyses to date, although small amounts have been found at times. It is known that manganese concentration is highest in the bottom strata of the reservoir during stagnation when it goes into solution with carbonic and organic acids and is afterwards mixed with the upper water during the turnover periods. It seems likely that sufficient manganese to cause trouble was not present prior to the fall of 1929. These observations confirm the experience at certain other water purification plants that have had similar occurrences of manganese in their reservoirs.

COPPERAS-LIME TREATMENT

Treatment designed to eliminate manganese from the filtered water and thereby prevent subsequent precipitation in the mains was not deemed practicable with the use of alum as a coagulant in view of the high pH that is required to precipitate manganese prior to filtration. In plants where lime is used for softening with a resulting pH of 11, or thereabouts, and recarbonation follows coagulation and settling, alum can be used satisfactorily and manganese removal becomes incidental to the softening process, but such a procedure with soft water would involve extra expense for lime and recarbonation merely for the sake of using alum. Of the other possible coagulants on the commercial market, ferrous sulphate or "copperas," offered itself as the most economical and readily obtainable and required no changes in equipment for immediate use. After some preliminary laboratory experiments to determine, in so far as possible, the optimum copperas and lime doses, the copperas-lime treatment was initiated on a plant scale in August, 1931.

Raw water was drawn from the lower intake, near the bottom of the reservoir, where the water is from 70 to 85 feet deep, the iron content being 1.0 p.p.m. and manganese 0.35 p.p.m. As stagnation effects became more pronounced during the fall the iron increased to 3.1 p.p.m. and the manganese to 0.75 p.p.m. With this increase in coagulants, provided by nature, the required dosage of copperas was greatly reduced, decreasing from 0.41 the middle of August to 0.16 g.p.g. by the first of October. With the fall turnover of the water, the iron dropped to 0.4 p.p.m. and the advantage that had been obtained by using water partially coagulated by natural forces was lost so that it was necessary to raise the copperas dose to 1.3 g.p.g. During the past winter, which was extremely mild with practically no ice formation on the reservoir the raw water has been in a more or less

constant state of turnover and no stagnation effects have been discernible, with the result that the copperas requirement has been about 1.4 g.p.g. since the beginning of December. Inasmuch as the total amount of coagulant which must be present, (iron in the raw water plus iron in the chemical dosage), for proper coagulation is less during the period when the carbonic acid content is greatest as a result of stagnation, the amount of alkali needed has been much more constant; the proper dosage of hydrated lime varying between 1.1 and 1.25 g.p.g. throughout a seven months period, which has probably included most of the variations in the character of raw water that are likely to be encountered in the future.

In addition to a reduction in coagulant requirements, the continual use of the bottom water from the reservoir maintains a fairly uniform low temperature in the filtered supply and avoids the use of water containing taste and odor producing algae such as are found in the upper waters of the reservoir during warm months.

The copperas is applied by gravity feed to the raw water which is then immediately sprayed through the influent aerator. Following aeration the water flows by gravity through a rectangular, concrete conduit to the mixer, sufficient hydrated lime being applied 270 feet from the mixer to raise the pH to approximately 10. Under present usual rates of operation, 20 to 45 m.g.d., velocities in the conduit vary from 0.37 to 0.84 feet per second with an increase to 2.0 and 4.5 feet per second, respectively, as the water enters the mixer. The mixer is of the circular, tangential flow type with a diameter of 35 feet and 30 feet deep, which provides mixing periods varying from 6 to 15 minutes at the several rates of flow. Operation at the lower rate of flow, which is not sufficient to meet the demand over a very extended period, provides a much longer mix, but reduces velocities proportionately, thereby nullifying the benefit of the increased time.

Water flows from the top of the mixer into the smaller of the two large settling basins which are principally formed by natural topography and have a combined capacity of 125 million gallons, permitting operation in series, as now used, or in parallel. Each of these basins is provided with a baffle and the actual time of retention of the coagulated water is about two days. The depth of the second, or larger of the coagulation and settling basins near the outlet point is over 20 feet and settled water passes over a submerged weir near the surface into the conduit to the filters. Filters are of the rapid sand type, and are followed by effluent aeration as the final step in the purification process, although chlorination may be used, if needed.

COAGULATION

Water drawn from the lower intake of the Scituate Reservoir requires the addition of less coagulant and better coagulation results from its use than from the use of water nearer the surface, in which oxidation is more complete. As previously mentioned, the advantage gained from using such water, containing considerable quantities of natural coagulants, disappears with the fall overturn, and winter stagnation is not of sufficient magnitude to produce appreciable effects on coagulation; nevertheless, it has been possible by proper increase and control of chemical dosing to produce an effluent during the winter months fully equal in quality to that which was produced during the fall.

The copperas dosages required were mentioned previously, varying from 0.16 to 1.5 g.p.g. The proper iron dosage can be determined only by experimentation on the water to be treated, as it varies considerably at different seasons, but the lime dosage necessary for proper coagulation under the physical conditions existing in any one plant will bear a definite relation to the total amount of iron present during coagulation and to the carbonic and other acid content of the water which is being dosed. At Providence, lime used by the reaction over a period of four months has been equal to that necessary to neutralize acid in the aerated water plus 2.1 mols of (OH) per mol of (Fe) present in the water during mixing; including the (Fe) in the raw water and the (Fe) added as coagulant. Lime added in excess of this requirement produced causticity in the coagulated water. Iron in ferric form, when used for coagulation, requires less alkali for the reaction and will coagulate at a lower pH than that necessary for the oxidation of ferrous iron.

Precipitation of manganese took place in the distribution system, as already mentioned, at a pH of about 8, but the reaction at such a pH is altogether too slow for results on a plant basis. Manganese oxidation requires a pH of about 9.2, and so, while ferric iron in various forms will coagulate at much lower pH values, the removal of manganese requires that the minimum pH shall be not less than 9. Oxidation of ferrous iron within periods necessary to produce good coagulation requires a pH considerably above this point, but dependent to some extent upon the physical treatment to which the water is subjected, such as mixing and sedimentation.

The relation of points of application of chemicals in the iron-lime

process is of utmost importance in a soft water; if applied wrongly a solution may result, which might be termed a colloidal dispersion, from which there will be no visible precipitation within a week or more. Experiments on a plant scale have shown that iron must be applied first and a certain period of contact allowed for hydrolization of the iron compound before the addition of alkali. The length of contact period which is necessary, or desirable, varies with the condition of the water being treated. The presence of alkalinity during this period is not necessary for successful operation, which is distinctly at variance with the commonly accepted belief that alkalinity sufficient to react with the iron coagulant must be present. The alkalinity of the raw water during the winter has been less than fifty percent of the calculated requirement.

Other writers have called attention to the fact that more is dependent upon the physical forces in the formation of iron floc than is the case with alum. This has proved true at Providence where the absence of turbidity is in marked contrast to the condition in those plants where iron-lime treatment has been considered practicable. Lack of turbidity causes slower action and with the mixing facilities available, which were ample with alum coagulation, the floc formed with iron has been very fine in size. Fine floc demands a long period of subsidence, which is provided at this plant and produces satisfactory results; but even after the two-day period for settling, loading of the filters is considerable due to fine particles of floc carried onto the filters. Slow settling of the floc has two advantages, however, in that the action of adsorption and the physical sweeping-out action are at a maximum, although at the high pH used 5 to 8 parts per million of color are so highly peptized that their removal is impossible with rapid sand filtration. Increased mixing would result in a coarser floc which would settle much more quickly, making the treatment applicable to plants with comparatively short coagulation periods. There is a possibility also that with slight mixing an upward flow contact filter might serve to replace settling basins with this process. If the ferrous form of iron is used for a coagulant, there is a certain percentage of iron which is not oxidized under plant conditions and this will not be included in nor adsorbed by floc which forms and settles too quickly. Floc may be formed from coagulants supplied in the ferric state much more quickly than with copperas and with excellent results. Iron in ferric form, such as the chloride, is available in certain localities at reasonable cost at the present time and in other

localities the ferrous iron may be obtained and converted to ferric, if so desired, by chlorination, without undue cost of chemicals.

Manganese has been removed continuously to 0.02 p.p.m. in the effluent and during the past four months the removal has been absolute, tests failing to show any indication of its presence when testing for one-hundredth of a part per million. With 0.75 p.p.m. of manganese (Mn) present in the raw water the effluent contained only 0.02 p.p.m. The percentage of the manganese removed during the subsidence of floc in the basins has varied considerably and has not remained the same as the proportion of iron dropped between the mixing process and entrance to the filters. This suggests that much may depend upon the state in which the manganese exists in the raw water. However, removal by the filters was practically complete in those periods when only a small percentage of the manganese was removed in the settling basins.

FILTRATION

The ten rapid sand filters were designed to give the plant a capacity of 48 m.g.d. Each filter is built in two independently operated sections, 55 feet by 16 feet. The beds are composed of 6 inches of stone, (1 to 2 inches in diameter), 12 inches of gravel and 30 inches of sand with an effective size of 0.37 mm. and a uniformity coefficient of 1.46 when placed. After five years of operation with alum treatment the effective size had increased to 0.42 mm., the uniformity coefficient remaining about the same with a decrease in depth to 27 inches due to losses by washing.

The very finely divided floc produced with alum treatment necessitated careful supervision in handling the filters which have always remained in good condition. Filters were washed for about 4 minutes at a 7-foot loss of head, although 10 feet were available, wash rates varying from 14 to 27 inches rise of water per minute, depending on the water temperature. Maximum expansion lasts for only two minutes, this being the period during which the hydraulically operated gates are fully open.

With the introduction of copperas-lime treatment, the filters began to receive a settled water having a pH of 9.8 to 10.0 in contrast to the settled water under the alum treatment which had a pH of only 6.2. With this radical change in the hydrogen-ion concentration organic matter which had collected in the filters became highly peptized, increasing the color of the filtered water from near zero to as much as

55 p.p.m. Cleaning the sand with caustic soda solutions varying from 1.0 to 1.4 percent in strength proved very effective in removing the organic matter and by the end of August all filters had been cleaned and were producing a water with a color of 12 p.p.m.

Cleaning the filters

The method employed in cleaning the filters may be of interest since it was somewhat at variance with common practice in that rather weak solutions of caustic were used, thereby considerably reducing the cost of chemical. The filters were washed with water at the usual rates after which the water level was drawn down to 18 inches over the sand. Flake caustic soda was then dumped into the water in sufficient quantity to produce solutions varying in strength from 1.0 to 1.4 percent sodium hydroxide. The sand surface was next raked with asphalt rakes to break up the surface mat, raking being repeated after the solution had been drawn down into the sand about six inches. The caustic solution was drawn down further into the filter at intervals of four hours in order to insure maximum action of the caustic near the surface of the sand, since this zone contained the greatest accumulation of organic matter. This procedure made it possible to obtain practically complete utilization of the caustic solution. After a period of 20 to 24 hours the filter was washed as in ordinary operation and then, while sustaining a wash water flow of 12 inches per minute, the bed was raked thoroughly. This was accomplished by men standing on the gutters and using long-handled rakes with which it was possible to reach to the gravel surface and break up all lumps that were encountered throughout the entire depth of the sand bed. Raking required approximately one-half hour for four men on each half of a filter, and was followed by a wash at ordinary rates. Filtering to waste was then started and continued until the effluent water was free from causticity and a satisfactory color was being produced. The method of cleaning, as outlined, proved very effective, with a slight advantage noted for the 1.4 percent caustic solution over the weaker one.

Since cleaning, operation of the filters has been satisfactory and, except for a period during the fall turnover when loading of the filters was materially increased due to improper control of the coagulation process, because of so limited experience with the new treatment, an effluent of desirable quality has been produced. The length of filter runs is comparable with the average for the same nine months of the

year previous when alum was being used. Difficulties experienced in the proper coagulation of a water are always reflected directly in the operation of the filters and so we find that during the period just prior to the turnover in the reservoir, while coagulation was the best, filter runs increased to 48 hours, but immediately following the mixture of the top and bottom waters in the reservoir the loading of the filters became such that runs decreased to slightly less than twenty hours for a short time.

These fluctuations in length of runs were, of course, duplicated in inverse ratio by the percentage of wash water required. Washing was performed as in previous years during the first few months of the new treatment, but later the time of wash was increased one minute to give a total time of five minutes. The rate of wash was also increased as tests indicated that a greater rate of flow was necessary to clean the sand properly. A slight increase in the vertical rise per minute was, of course, required to produce the same expansion after cleaning the sand with caustic, but proper removal of the iron floc demands a flow sufficient to produce a somewhat greater expansion than was needed with the alum floc, probably due to the greater density of the iron floc. Washing has required 2.5 percent of the total filtered or approximately 20 percent more than with alum coagulation during the previous year.

Accumulation of iron and organic matter has not been sufficient to cause any trouble. Before the increase in rate of wash, mentioned above, there was a slight tendency towards accumulation and some mudballs were formed, but since the modification in the wash these troubles have disappeared and the sand has a clean appearance below the surface penetration layer which is visible for a depth of about six inches.

Table 1 shows a recent analysis on sand removed from a filter at the end of a run and before washing.

The results given in table 1 show rather conclusively that the coating of sand grains has been negligible and that while there is some adsorption of iron at depths of 12 and 18 inches there is no accumulation at those depths. Analyses of the filter effluents have shown unvarying characteristics and there has been no evidence of any tendency towards fluctuations due to decomposition or disintegration of deposits in the lower levels of the filters such as has been found in some plants.

DISCUSSION

In considering the advantages offered by the new treatment, the beneficial results should be tempered with some consideration of the difference in characteristics of the present effluent as compared with the effluent produced with alum coagulation. The color has increased approximately 6 p.p.m. and the soap hardness has jumped from 18 to 38 p.p.m. Except for a few industrial users who object to the increased calcium content, practically no dissatisfaction with the change has been expressed, and so far, domestic consumers appear to have found nothing objectionable about the quality of the new water. In contrast to the somewhat undesirable features above noted is the marked decrease in complaints from consumers since the new treatment was put into effect. The extent to which corrosion has been reduced is reflected in table 2 which shows that only one-seventh as many complaints have been received since using the copperas-lime treatment, as in the same months of the previous year.

TABLE 1

Accumulations on sand in filter 8-B, April 5, 1932

(Results expressed as percentage of total weight of air-dried sample)

	DEPTH BELOW SURFACE, INCHES				
	0 to 1	1 to 2	6	12	18
Organic and volatile	2.1	1.4	0.6	0.4	0.3
Iron (Fe_2O_3)	1.03	0.60	0.54	0.17	0.17
Manganese (Mn_2O_3)	0.03	0.03	0.01	0.01	0.01
Undetermined	0.04	0.27	0.25	0.22	
Total accumulation*	3.20	2.30	1.40	0.80	

* The total amount removed from samples by 1:3 hydrochloric acid in 24 hours.

So far, the effluent of this treatment is stable, there being no indication of any precipitation in or solution from the pipes. Daily tests of the plant effluent and of water from taps in the distribution system show results that are so nearly identical as to completely substantiate such a statement. Corrosion of pipes has been reduced to a minimum as is evidenced by the fact that all bleeders were shut off early in November, 1931, and since that time complaints have been received from consumers on only two of the dead-ends.

TABLE 2

Summary of all complaints received, regardless of cause, since December, 1929

	1929	1930	1931	1932
January		58	59	4
February		51	43	2
March		55	33	4
April		40	12	
May		33	19	
June		40	16	
July		11	29	
August		22	8*	
September		16	5	
October		22	7	
November		30	10	
December	115	87	8	
Total		465	249	

* Treatment changed from alum to copperas-lime on August 9, 1931.

TABLE 3

Characteristics of plant effluent and tap water in distribution system—average for three months period

JANUARY 1 TO MARCH 31	1931*		1932†	
	Plant effluent	Tap	Plant effluent	Tap
Color	2	7	9	9
pH	9.1	9.0	9.7	9.7
Alkalinity (methyl orange)	7.3	7.4	18.6	18.7
Alkalinity (phenolphthalein)	1.7	1.7	10.7	10.4
Soap hardness	18.3	18.3	37.7	37.4
Turbidity	0.0	0.0	0.0	0.0
Iron (Fe)	0.03	0.07	0.11	0.11
Manganese (Mn)	0.18	0.14	0.00	0.00
Alumina (Al_2O_3)	0.45		0.26	
Silica (SiO_2)		6.32		7.24
Calcium (Ca)		5.94		16.07
Magnesium (Mg)		0.40		0.43
Sulphate (SO_4)		11.54		11.25

All results, except pH, expressed as p.p.m.

* Alum coagulation with lime added after filtration.

† Copperas-lime coagulation.

Table 3 compares the characteristics of the effluent of the plant during the months of January to March in 1931, using alum coagulation with the same months in 1932, using copperas-lime coagulation.

HANDLING OF CHEMICALS

The copperas being used at Providence is the best grade of commercial granular copperas obtainable and is received in 200-pound sacks in carload lots of 20 tons at a point on the railroad about 2 miles distant and hauled by truck to the plant. The sacks are emptied into a bucket conveyor and elevated to the concrete storage bins of which there are three. The bins, rectangular in section with pyramidal bottoms, have a depth of 16 feet and discharge through a 7-inch pipe at the bottom. Each bin has a capacity of 1427 cubic feet and will hold slightly over 40 tons of granular copperas which constitutes a fifteen days supply at the present rate of use.

Considerable difficulty has been experienced with caking in these bins and even in the weighing hoppers above the dry feed machines, with occasional stoppage in the machines caused by lumps plugging the entrance to the helical-type feeding screws. Although this caking or cementing is never strong enough so that bits of the cake cannot be crumbled easily in the hand, it is, nevertheless, a rather hard and unpleasant task to start the flow in a bin which contains 20 to 40 tons of copperas in one large lump.

Lime used to date has been the commercial hydrate, (at least 90 percent available $Ca(OH)_2$), which is delivered at the Purification Works by truck in paper bags containing 50 pounds each. The lime is fed through screw feed machines into conical hoppers and washed into water ejectors which feed the milk of lime into a conduit under approximately 20 feet of head.

The lime is stored in the chemical feed house, originally designed to store lime sufficient only for final pH correction after filtration and storage space is therefore decidedly inadequate for the present requirement and remodelling of facilities will be required for the most economical operation. There is a possibility that when this is done facilities will be provided suitable for the use of oxide of lime, rather than the hydrate, since a saving in chemical cost of approximately 25 cents per million gallons of water treated would be thus effected.

EXPERIMENTAL WORK

Experiments on a laboratory scale indicate better coagulation and settling with ferric iron coagulants than is obtained with the ferrous

coagulants, the amount of iron (Fe) required being apparently the same regardless of the ferric compound used.

SUMMARY AND CONCLUSIONS

1. Manganese is completely removed from the soft, colored raw water at Providence by copperas and lime treatment, maintaining a pH of about 10 for coagulation.

2. The treatment has stopped red water and manganese troubles in the distribution system and eliminated the necessity of using bleeders or dead-ends.

3. Increases in color, hardness and iron in the effluent over the amounts contained in the effluent under alum treatment have not been of sufficient magnitude to cause concern to any except some industrial users who note the hardness change.

4. The use of bottom water from the reservoir has been helpful to coagulation during stagnation periods. It produces a water of fairly uniform low temperature throughout the year. It has so far been free at all times from tastes and odors such as are present in the upper waters due to algae.

5. Filter operation is satisfactory and no trouble has been experienced in keeping the filters clean. Analysis show that the coating of sand grains has been negligible in eight months of copperas-lime treatment.

6. Experience at Providence has been that granular copperas will cake when stored in bins or bags and tends to introduce dosing irregularities with dry feed equipment.

7. Experiments indicate better coagulation and settling with ferric coagulants than with those in the ferrous form.

DISCUSSION

EDWARD S. HOPKINS:[6] As shown by published papers, the successful use of iron and lime for the removal of manganese has been in vogue at this plant for many years. The initial control of this coagulation was to maintain an alkalinity in the mixing basin just pink to phenolphthalein. With the introduction of pH control, it was demonstrated that an alkalinity equivalent to a value of 9.0 was necessary to produce maximum floculation with iron and lime.[7] Later investigations indicated that manganese could be *entirely* removed by iron

[6] Principal Sanitary Chemist, Montebello Filters, Baltimore, Md.

[7] Hopkins, Ind. Eng. Chem., 21: 58, 1929.

coagulation at the same pH value, namely, 9.0.[8] Studies with ferric and ferrous salts gave similar results, indicating that the phenomenon is selective surface adsorption by the hydrated ferric oxide floc.

Soluble manganese salts may be converted to the hydrated oxide by the action of air or dissolved oxygen in the water.[9] Reaction with alkali produces this compound. It is believed that coagulation with iron and lime at the high pH values proceeds in two systems. As stated, the manganese is adsorbed by the hydrated ferric oxide, but since there is present hydrated manganous oxide also, this substance adsorbs colloidal manganese particles, giving a double value to the treatment. Of course, the ratio of iron floc to manganese floc is much greater; therefore, most of the work is done by the iron. The adsorption of the hydrated manganous oxide floc explains the action of sand, gravel, or coke trickling bed removal in conjunction with aeration treatments.

It is stated in this paper the manganese was precipitated when the pH was raised to 9.0. This is comparable to results obtained at Luke, Maryland. Patrick[10] maintained a pH below 8.5 keeping the manganese in solution, which if raised above that value is precipitated in the mains. Since pH 9.0 is approximately the formation point of hydrated manganous oxide, deposit would be expected from waters containing this element under these conditions.

Operating experience at Montebello endorses the necessity for increased mixing periods when iron and lime coagulation is used. It is believed that complete ionization and oxidation of the ferrous salt is necessary before addition of the alkali. Excessive contact time is not required for this purpose. Incomplete mixing after addition of the alkali will produce pin head floc or other types of poor coagulation. If sufficient contact time is given before addition of the alkali, all of the iron from the copperas will be oxidized and converted to the hydrated oxide by the lime. Complete removal of iron is possible under plant conditions using ferrous sulfate with lime as the alkali and with coagulation at pH 9.2. The ferric salts not needing subsequent oxidation, can be utilized with a lesser mixing period or lower rate than the ferrous salt.

The situation as presented in this paper clearly shows the value of hydrated ferrous oxide for manganese adsorption. When sufficient

[8] Hopkins, Ind. Eng. Chem., 22: 79, 1930.
[9] Hopkins, Ind. Eng. Chem., 19: 744, 1927.
[10] Patrick, J. G.: Private communication.

mixing is available, the utilization of ferrous or ferric salts is of equal value. If limited mixing is obtainable, the ferric salt is more efficient, since it is not necessary to give a contact period before addition of alkali. This paper once again demonstrates that precipitation of hydrated ferric oxide floc at pH values above 9.0 will completely remove soluble manganese from a water supply.

PAUL W. FRISK[11] The paper on "Iron and Lime Treatment for the Removal of Manganese" has been well presented and must be acknowledge with a great deal of credit.

Where the raw water is always low in calcium salts, the application of lime will tend to have the chemical go into the solution, and consequently increases the hardness of the water. Mr. C. P. Hoover in a paper entitled "*Reduction of Carbonate Hardness by Lime Softening to the Theoretical Limits*" gives the solubility of calcium carbonate as 13–20 p.p.m. and of magnesium hydroxide, 1 p.p.m. Furthermore, the use of lime requires very careful dosage and control, in order to obtain the maximum precipitation of calcium carbonate.

For the elimination of corrosion, the use of lime serves this purpose most efficiently, although the Lime must be added to the effluent.

The most modern types of textile industries provide the distribution of their water through redwood stave lines, brass or cement lined pipes. When these kinds of pipe are used, red water troubles are eliminated and corrosion factors are considerably reduced.

The water of North Carolina is particularly low in calcium and magnesium salts, and therefore necessitates different types of treatment for the elimination of manganese. In many of the waters used in textile industries, the treatment was either accomplished by the common alum-soda-chlorine treatment or zeolite; and for rayon industries other more elaborate processes give a finished water which is more suitable for the process.

The bleaching of many textile materials by the use of sodium hypochlorite requires a water nearly free from calcium, magnesium, iron and manganese salts, and more particularly is this true in the finishing bath. Waters moderately high in these constituents precipitate on the surface of the bleaching solutions, and with calcium and magnesium form insoluble soaps which affect the final quality of the yarn.

Other types of treatment for the removal of iron, manganese, calcium and magnesium salts, which have been worked out by the Rayon

[11] Chemical Engineer, American Euka Corporation, Euka, N. C.

and Textile Industries, are shown below. The first has met with a great deal of approval and has been found to be very practical where the hardness of the water is low.

1. The use of aluminum sulphate 18 aqua, magnesium chloride 6 aqua and caustic soda may be compared with the treatment with sodium aluminate-caustic soda and aluminum sulphate-sodium aluminate-soda ash-caustic soda.
2. Ferrous sulphate, potassium permanganate and caustic soda.
3. Manganous sulphate and caustic soda.
4. Ferric sulphate 7 aqua, magnesium chloride 6 aqua and caustic soda.

These four types of treatment will all eliminate iron and manganese and reduce the total hardness down to about 4 p.p.m.

They will prevent corrosion due to the fact that they eliminate all of the free CO_2 and when sufficient alkali is used coat the inside of the pipe lines with calcium carbonate. The coagulation on the above types of treatment is not affected by the temperature, and can be used to great advantage in climate where the weather is cold during part of the year. The floc is usually heavier, facilitating the settling time and making possible the use of the Dorr Clarifiers for removing sludge.

UNSUSPECTED, UNUSUAL AND LITTLE KNOWN FACTORS IN WATER SUPPLY QUALITY

By Paul Hansen[1]

Until a very few years ago water works engineers and sanitarians centered their energies on the application of weapons known to be effective in a wholesale way against the scourge of typhoid fever, cholera and some other food and drink borne diseases. Among the principal of these weapons were filtration of public water supplies, pasteurization of milk supplies and typhoid inoculation.

The first of these in the field and unquestionably the most important and effective was filtration of public water supplies. Its value was appreciated by a few as early as 1885 (1) but it was not until the dramatic demonstration of the efficacy of filtration in 1892 when Hamburg suffered a severe epidemic of water borne cholera, while the adjoining city of Altona was saved by filtration.

From this time on public water supplies were improved by filtration and later by disinfection (with and without filtration) at an increasingly rapid rate with the result that at the present there are very few water supplies obviously liable to pollution that are not receiving systematic treatment.

The effect has been a continuous reduction in deaths from typhoid fever during the thirty years of active expansion in purification of public water supplies. Perhaps all of this reduction cannot be ascribed to water supply improvement, some of it undoubtedly is due to pasteurization of milk supplies and to anti-typhoid inoculation, but there is evidence in the statistical data that safe public water supplies have had by far the major influence.

LESSER DANGERS OBSCURED BY GREATER DANGERS

During all this period the lesser dangers and imperfections in public water supplies were given comparatively little consideration in the eagerness to block quickly the main avenues through which poured preventable sickness and death. In the face of the major

[1] Of Pearse, Greeley and Hansen, Chicago, Ill.

evil the minor evils were obscured. Moreover the earlier barriers against pollution in water supplies were relatively crude and did not exclude all imperfections or even all water borne disease.

With more general acceptance and application of the purification of public water supplies there developed a greater alertness to the presence of water borne diseases; prevalence and epidemics of typhoid and enteritis were more thoroughly investigated and in course of time it was revealed that filtration was not always dependable and above all that there were unsuspected, unusual and little known factors in water supply quality.

LESSER DANGERS INCREASINGLY RECOGNIZED

Year by year the list of these factors is being added to. Year by year they are being more effectively guarded against. Year by year we take greater cognizance of the minor and more subtle factors influencing quality. Year by year our judgment of what constitutes a satisfactory water supply service becomes more rigorous.

During the past few years water works men have become conscious of the fact that we have passed from the pioneer period of establishing filtration and disinfection as a necessity against disease to the developmental period of refinement and perfection of processes. In other words, the battle is over and "mopping up" is under way.

LOWELL TYPHOID EPIDEMIC OF 1903 SOUNDS WARNING AGAINST CROSS CONNECTION

The first startling realization of the necessity of adequately protecting a water supply after it had been delivered in the mains came in the form of an epidemic of typhoid fever at Lowell, Mass. in 1903 (2) due to a cross connection between the city mains and factory water supply mains for fire fighting, which latter was supplied with water directly from the polluted Merrimac River. This opened the war on cross connections which has given this Association many an exciting meeting until about 1925 and a final unanimity of opinion between fire insurance interests and sanitarians has even yet not been reached. However, safeguards are now applied to cross connections which greatly reduc the danger, but do not absolutely eliminate it.

As usual a number of epidemics resulting from connections between public supply mains and mains of private supplies of inferior quality, were necessary to secure general recognition of the danger. Among

the more notable of these are the outbreaks at Winona Lake, Indiana in 1925, at Everett, Washington in 1923, at Fort Wayne in 1923 and 1929, at Albany, N. Y. in 1917 and Philadelphia in 1913 (3).

MORE CROSS CONNECTIONS

Another cross connection danger developed within factories, hotels and other buildings where there existed pipes carrying the public water supply and other pipes carrying polluted water. Sometimes these cross connections were installed maliciously to steal water, while at other times they were installed more or less innocently by an employee without the knowledge of the management. There are a number of epidemics due to this cause of which the most notable are those at Bloomington, Illinois in 1920, Franklin Furnace, N. J. in 1922, Cohoes, N. Y. in 1927, and Circleville, Ohio in 1914.

DEFECTIVE BARRIERS

An epidemic of typhoid fever at Salem, Ohio in 1926 (4) is an illustration of the danger of pollution of gravity conduits not made absolutely tight. This conduit happened to be a vitrified tile with bad joints, but the principal applies equally to any pipe of any material which may be subjected to a greater head of water on the outside than on the inside. An epidemic of enteritis at Greenville, Illinois (5) was caused in the same way. The Chicago epidemic of 1923 belongs in the same category. There are many others.

Ferguson and Klassen (6) described a number of unusual instances in which water supplies have become polluted. Among these are the following:

Entrance of oil and other surface pollution into the top of well casings not properly protected.

Entrance of surface and subsurface pollution through corroded and broken casings.

Passage of pollution down the outside of casings and into the well through imperfect "packers" or seals to exclude surface water as at Lockport, Illinois.

Passage of polluted water from a sewer, privy, river or canal through leaks in reservoir walls especially during periods when the reservoir is drawn low. Such cases occurred at Marseilles and Rockford.

Entrance of polluted roof water into a reservoir through an overflow pipe due to clogging beyond the junction of the roof water pipe and the overflow pipe.

Pollution of open reservoirs, stand pipes and tanks by birds and wind blown dirt.

Siphonage or back flow from roof tanks subject to pollutions. To prevent this the inlet pipe should have a free discharge into the tank above high water level. This is especially important when the tank is also supplied from an additional source less pure such as auxiliary fire fighting supply.

Pollution entering reservoirs having leaky bottoms or having a natural earth bottom subject to pressure or more or less polluted shallow ground water on the outside.

To these may be added danger of connecting reservoir drains or blowoffs with nearby sewers which may during storms flow under pressure sufficient to cause outside pressure against the control valve.

SIPHONAGE AND BACKFLOW

Gorman (7) and Connelly (8) have demonstrated dangers from siphoning of plumbing fixtures, hospital equipment containing infectious matter and swimming pools where pressure is drawn low on the distribution mains by fire demands or other heavy drafts. These dangers are especially great on upper floors.

At Lake Forest, Illinois, there are a number of large estates, many with swimming pools. Some of these have 8-inch connections and are provided with booster pumps. On hot days in the summer the owners of these are often moved by a simultaneous desire to fill their swimming pools and otherwise use large quantities of water with the result that a negative pressure is placed on the mains tending to draw in soil pollution through any leaky joints that may exist. No disastrous results have thus far come about in this way, but the danger exists nevertheless.

NEWLY LAID MAINS, PACKING AND JUTE

The possibility of pollution as a result of filth left in newly laid mains is well known and at present generally effectively guarded against by heavy chlorination following the laying of pipe. Spaulding (9) demonstrated that jute used in making bell and spigot joints may be heavily infected with the colon bacillus causing unfavorable water analyses for long periods. The same is true of some pump packing also described by Spaulding (10).

The difficulty may be largely overcome by use of sterilized jute which is more expensive but equally satisfactory for use. It is known

that the preparation of jute is a rather filthy operation and it is conceivable, but not yet demonstrated, that the jute may contain organisms capable of producing intestinal disturbances.

FILTER PLANT DEFECTS

The foregoing instances of possibilities of accidental and unlooked for pollution constitute by far the largest class of special dangers to public water supply. The next class is a relatively small class comprising filter plant defects, but has given purification plant operators much trouble. Some result from faulty design or construction and are more or less beyond the control of the operator. Others are due to faults or accidents in operation. Some filter plants are built with a common wall separating the filtered water reservoir and the sedimentation basins. Should a leak develop in this wall it is certain to allow polluted water to enter the clear water reservoir because the level of the water is always at least ten feet higher in the sedimentation basins than in the clear water reservoir. Tests for water tightness following construction do not afford adequate assurance because sometimes leaks develop months and even years after the plant is placed in service.

Formerly it was a custom to discharge roof drainage into filters or sedimentation basins. A few experiences of this sort where bituminous roofs were used was sufficient to stop this practice fully twenty years ago, but an occasional inexperienced designer of the newer generation jumps to the conclusion that this represents a cheap and satisfactory way to get rid of roof water.

Wooden roofs over filtered water reservoirs are sometimes creosoted. This is permissible if one makes certain that the timbers will no longer drip creosote. Otherwise, troublesome tastes will be encountered for quite a long time. This condition occurred in a small filter plant in Kentucky recently. The taste was satisfactorily overcome by use of ammonium prior to chlorination as the water flowed into the clear water reservoir.

Sometimes it is found convenient to pass the wash water sewer or other pipes carrying polluted water through the clear water reservoir. This is bad practice as a leak may develop in these pipes without being readily detected. In short it may be set down as a definite rule that any pipe or structure containing polluted water should be separated from the filtered water reservoir by a clear space that may be inspected and that can be drained to prevent building up a head of water greater than exists in the filtered water reservoir.

Recently at Winnetka, Illinois, the operator had occasion to empty one of the sedimentation basins before there was a sufficient accumulation of sediment to warrant cleaning. Accordingly the sediment was not removed but permitted to partly dry and decompose for a period of about a week. When water was readmitted to the basins the sediment was stirred up and imparted a pronounced and disagreeable taste to the filtered water.

Filters out of service for a period of time, especially if the sand is heavily coated with organisms, may impart tastes and odors to the filtered water and bacterial counts in the filtered water may rise greatly.

Careless use of wash water valves may result in disturbance of the gravel in the bottom of a filter permitting sand to escape into the filtered water reservoir and by greatly thinning the sand in places permit passage of improperly purified water.

Uncovered openings or other ready access to filtered water reservoirs should be avoided as they invite careless or even malicious pollution of water. Even where reservoirs are covered some precaution is necessary. Recently during an enlargement to an existing filter plant and before sanitary facilities had been provided by the contractor, a secluded corner of the top of a concrete and earth covered filtered water reservoir was improperly used by the workman. Some pollution entered the reservoir through a very small crack and was sufficient to cause a slight rise in the colon content of the water in the reservoir.

There are of course many things that a careless or incompetent operator of a filtration plant can do or leave undone that will injure the quality of water as delivered into the distribution system, but this leads into the question of operation which is outside the scope of this paper.

SUDDEN OR UNLOOKED FOR CHANGES IN POLLUTION OF SOURCE OF SUPPLY

Sudden changes in the pollution of a source of supply may adversely affect the quality of water as delivered to consumers before the operator has time to apply corrective or preventive measures or it may be that corrective or preventive measures are not available or possible.

Sudden and often unmanageable pollution caused by phenol wastes is now an old story. Pre-ammoniation has proved a valuable

preventive of chlorophenol tastes, but is not effective when phenol pollution is strong enough to create a taste by itself. Activated carbon, however, seems capable of removing considerable concentration of phenols.

High color in water is sometimes a troublesome factor. Quantities of alum required for color removal are sometimes excessive. The Minneapolis water supply is the outstanding example of a supply affected from time to time with excessive color. Where color in water supply is uniformly high as in some New England localities, the public becomes used to them.

In recent years manganese has become more prominent as a trouble maker in purification plants often of sudden occurrence and sometimes to the extent of seriously clogging filter underdrain systems. Redissolving of manganese in idle filters may be troublesome as reported by Janzig and Montauk at Minneapolis (11).

DROUGHT EFFECTS

Intensification of both sewage and industrial pollution following prolonged drought may create great difficulties as at numerous places during the drought of 1930. Principal difficulties were in the form of increased tastes and odor, but the load of organic pollution required unusually careful filter plant operation and extra heavy disinfection (12).

The most startling result of the drought was a pronounced epidemic of enteritis at Charleston following heavy pollution of the raw water supply, although the water as delivered to consumers complied with all accepted analytical tests as to potability.

Similar intestinal disturbances were observed in downstream cities along Kanawa and Ohio rivers following the outbreak at Charleston. As reported by Veldee (13) these outbreaks had many of the characteristics of a germ disease but the weight of evidence suggested some decomposition products or vital secretions.

There arises the question in this connection as to whether heavy pollution of raw water will invariably cause these disturbances or whether products caused by specific organisms must be present. Imhoff (14) reports that repeated re-use and reclamation of the waters of the Ruhr is limited only by the increase in salt content. Goudey (15) in Los Angeles reports favorably on reclamation of sewage for water supply purposes with appropriate means. These observations suggest that the disturbing, decomposition product, secretion or

phage is not always present or that by proper means it may itself be destroyed and rendered harmless.

The experience of Charleston and downstream cities, however, was significant because it demonstrated that there may be a limit of raw water pollution beyond which present methods of purification are not adequate. The Charleston experience does not give an adequate measure of this pollution. While the dilution of sewage and other pollution as reported by Tisdale was small the dilution at downstream cities was much greater.

DOES A PURIFIED POLLUTED WATER PRODUCE DISEASE IMMUNITIES?

An interesting question is raised by this discussion, namely: Will a purified water containing soluble products of aquatic life create immunities in the populations using the water so that they will be less subject to water borne diseases? This question recalls the old belief that an unaccustomed water supply causes intestinal disturbances until one got used to it.

CHEMICAL SUBSTANCES AFFECTING HEALTH AND METABOLISM

Some chemical substances in minute quantities are necessary for human metabolism and proper functioning of glands. Some of these same substances may be harmful if present in too large quantities. Among these are iodine, fluorine, various vitamines and perhaps a number of others that have not been discovered. Most of these substances are containing in foods and some are found in drinking water.

The one that has excited the most interest is iodine which if deficient causes a type of goiter. The universal use of public water supply led to the proposal that in regions where iodine is deficient the deficiency be corrected by wholesale medication through the medium of the public water supply. This was done for several years at Rochester, N. Y. (16) and several other places undoubtedly with beneficial effect. Opinion among health authorities and water works men crystalized against the practice largely on the grounds that it was not the function of water works authorities to distribute medication so long as there are available other means and other agencies more especially devoted to the cure of human ills. The subject is still debatable and there is much to be said on both sides.

Fluorine in water supplies has been credited with causing mottled enamel of the teeth (17). The evidence on this point, while strong, is not conclusive. Minute quantities of fluorine seem to be beneficial.

Vitamines are products of vital process and would not ordinarily be expected to be present in an inorganic substance like water. However, the fauna and flora of water may carry vitamines in appreciable amounts. Following the idea of iodization we might develop some vitamines by the addition of proper fauna and flora should there develop sufficient popular demand. This line of thought suggests the deliberate addition of immunizing phages to water supplies.

POISONS IN WATER SUPPLIES

Poisons in water supplies have always been feared. In former years they gave more concern than much more dangerous human pollution. Natural waters are singularly free from poisonous substances. The only poisoning of any moment through public water supplies has been by lead dissolved from lead pipes. Even this is confined to safe waters more particularly in New England.

Other forms of poisoning are nevertheless possible and should be carefully guarded against. Among the metals zinc and copper have been known to have ill effects and there is the ever present danger of poisoning from industrial wastes containing cyanides, mercury chloride and arsenic compounds. Actually there are no recorded cases of such poisoning through public water supplies, but it is quite conceivable that "spills" of such substances above intakes may have serious consequences.

DEVIATIONS FROM CONSTANT PHYSICAL CHARACTERISTICS

An important element in keeping the public satisfied with a public water supply is maintaining its physical characteristics constant. Occasional very slight turbidities will excite more comment than a constant turbidity provided it is not excessive. Changes in the character and intensity of tastes and odors excite more comment than a constant taste of uniform character, provided it is not excessive. Even mild chlorine cocktails will be tolerated provided the intensity of the flavor is not varied.

In practice the best way to avoid variations is to seek complete elimination of turbidity, tastes and odors, although it is conceivable that we may some day reach the refinement of applying a slight measured taste and odor by way of making the water pleasant to drink and giving it distinction. Thus there may be chosen a "bouquet" for the water supply of a city with the same discrimination and good taste that a lady observes in selecting her perfume.

SUMMARY

From what has been said, it is apparent that public water supplies are beset with many hidden dangers and subtle interferences that keep the conscientious water works manager constantly on guard. To meet the approval of a temperamental and imaginative public the average modern water supply must be a modified and processed product delivered to the consumers bacterially pure and of a constant or uniform quality. Perhaps we may use it to immunize, cure and regulate bodily process. Perhaps we may render it palatable with delicate flavorings. But all that is in the future.

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DISCUSSION

ABEL WOLMAN:[2] The paper by Mr. Hansen is so comprehensive in scope that it hardly seems necessary to make any comment thereon. It does appear valuable, however, to record one or two unusual examples of water borne disease, as characteristic of the kind of situation which arises from time to time and makes it necessary for the water works producer to be more than normally careful. As Mr. Hansen properly points out, now that mass typhoid fever generally does not occur, special precaution against intermittent disease caused by water supply becomes necessary.

In 1931, from August 10 to September 5, 126 known cases of Spirochaetal Jaundice appeared in Lisbon.[3] The total probable number of cases was in excess of 126. They were evidently of water origin and apparently were contracted by the drinking of a heavily contaminated source. This was the first appearance of this type of disease in Lisbon. Over 90 percent of the cases were in ages over 21, 74.9 percent were in males and 23.5 percent in females. The lethality was 24.5 per cent.

Fortunately the epidemic was detected early and was stopped before it had extended to pestilence proportions. It was largely restricted to one part of the city and to certain streets in that part, namely, the Anjos quarter in the northeast section.

The cases ranged around a public fountain in almost a perfect circle. The fountain is known as the "trench of angels." Practically all the cases showed a history of drinking from this source, although all the sources used were from calcareous broken strata.

Lisbon, in general, is supplied from a public water supply, but the people still use these fountain springs, which are supposedly endowed with mystical healing properties. Even the best pseople still use the water for the table and pilgrimages are made to the fountain for drinking each day. The public supply is chlorinated and its taste and odor at intermittent intervals also led to much use of the contaminated fountains.

On August 24, the "Angel" fountain supply was cut out of use and

[2] Editor-in-Chief, Journal of the American Water Works Association; Chief Engineer, Maryland Department of Health, Baltimore, Md.

[3] An Epidemic, at Lisbon, of Spirochaetal Jaundice of Water Origin Contracted by Mouth. Ricardo Jorge. Monthly Bulletin, International Office of Public Hygiene, 24: 1, January, 1932, p. 88.

the cases dropped rapidly, although the contamination of the fountain had probably already ceased before the discontinuance of its use.

The pipe line or collecting gallery supplying underground water to the "Angel" fountain was found to be broken and evidences of recent contamination by sewer rats were clearly established by the presence of dejecta from rats. The spirochaetes will not survive in water under a pH of 6.9. The present water was of pH 7.7. During the war, spirochaetes were most prevalent in calcareous regions and rare elsewhere. Apparently the rat plays the rôle of the reservoir of virus, while the water is a simple vector.

The transmission of unusual diseases by water has recently again been interestingly set forth by A. W. Burtt,[4] who discusses particularly the transmission of water borne diseases in swimming baths due to protozoa, worms and bacteria, where the skin and mouth are the vectors. In these instances, diseases are beginning to crop up in this country analogous to those which have occurred for many years in the far east, particularly where seasonal or perennial irrigation has been in use.

Mr. Hansen does well, therefore, to caution the profession once more that the watch-word of the water supply producer should always be "Safety First."

[4] Transmission of Disease by Water. A. W. Burtt. Municipal Engineering, 88: 711, 1931. Original presented before the Institute of Water Engineers of England.

BIOLOGICAL CONTROL AS AFFECTING PLANT OPERATION

By Carl Wilson[1]

The suggestion is often made that a modern complete treatment plant, embodying odor and taste removal by such methods as ammoniation or activated carbon, or both, makes it unnecessary to spend time and money upon control of plankton and other growths throughout the impounding system. At least upon the Pacific Coast this appears to be an erroneous assumption, and it seems reasonable to believe it just as wrong in the eastern or mid-continental sections of our country wherever water is stored for weeks or months before use. Where water is taken directly from a flowing stream, as at St. Louis for example, biological control above the intake is impracticable, but where water is stored for long periods, the task of treatment in the plant is lightened if reasonable control of growths is exercised.

For the purpose of the present discussion the meaning of the expression "biological control" has been extended to embrace not merely control of growths, but also of plant operation to meet changes in the character of the raw water caused by them, either while in the actively growing state, or through decomposition products of dead organisms, even though such plant control may be attained through chemical analysis. That is to say, both immediate and consequential effects of organic growth are included as integral parts of one problem.

ORGANIC LOAD

The presence of plankton organisms of any kind, algae, diatoms, or protozoans, implies concurrent reproduction, active life, and death, while finally, practically all organisms entering a treatment plant may be expected to die and disintegrate at an early stage in the treatment process. As a consequence of these vital processes the water acquires a load of organic substances of widely varying character,

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which, if not removed during treatment, must later be subject to bacterial attack with the certainty of producing objectionable odors and tastes if such bacterial activity occurs under anaerobic conditions. If allowed to enter the distribution lines these substances will stimulate heavy after-growths of bacteria as soon as the residual chlorine becomes exhausted, with consequent complaints of foul odor whenever the dissolved oxygen becomes depleted. Even in those cities where every effort is made to aerate water as it leaves the plant, anaerobic conditions are certain to be found in remote portions of the distribution system, or where consumption is light or infrequent.

TROUBLES WITH HIGH SULPHATES

Waters high in sulphate content not infrequently pass through a treatment plant and enter the distribution system in perfect condition, exhibiting no odor or taste, but at the far ends of the system are found to contain objectionable amounts of hydrogen sulphide. This phenomenon is due to the work of sulphate-reducing bacteria in the absence of oxygen. One California city using ground waters rich in hydrogen sulphide and soluble sulphates, had an interesting experience of this character. By combined aeration and chlorination (using dosages as high as 125 pounds of chlorine per million gallons) this water was finally sent into service absolutely free of hydrogen sulphide, and with a chlorine residual of 0.2 to 0.3 p.p.m. A few blocks away the chlorine was completely consumed and a mile or more from the plant hydrogen sulphide reappeared, increasing toward the remote end of the mains. Numberless complaints resulted, and one bathroom freshly painted in expensive white lead paint, was turned black after the first bath had been taken in the shiny new tub!

Sulphate-reducing bacteria cannot thrive however high sulphates may be unless organic matter is present in sufficient amount to serve as bacterial food, and also, through absorption of oxygen, to produce the anaerobic condition necessary for hydrogen sulphide to persist.

In regions where impounded sulphate waters are encountered, biological control directed toward reduction of the organic content of the raw water to the lowest possible point is especially important.

SELECTIVE DRAFT FROM IMPOUNDING RESERVOIRS

As a most effective means of preventing water spoilage in the distribution lines, a considerable expenditure of money to prevent the accumulation of organic matter in the raw water is thoroughly justifiable. Control of plankton growths, and to a greater or lesser degree, of marginal submerged plants, affords the most feasible means of reducing the dissolved organic content of surface waters.

The greater the amount of suspended matter present in raw water, the heavier the necessary dosage of coagulant, and also the more frequently filters must be backwashed. Moreover, if prechlorination is employed, the increased suspended matter (and turbidity in impounded water is generally due to organic growths, not to suspended silt) raises the chlorine demand, sometimes to a very high value. All of these effects of plankton growths increase the cost of plant operation through heavier consumption of chemicals and increased use of backwash pumps, and, where backwash water recovery is not practiced, the loss of water is a further expense. If all savings effected in these items are devoted to plankton control, we still have a further credit to the purification account due to avoidance of taste-producing potentialities in the finished water because of lessened organic content. Although it is difficult to assign specific monetary values to these savings, the advantages accruing from biological control of impounded raw waters are none the less real, and there is also a not-to-be-neglected value to the water purveyor resulting from the increased confidence and respect of consumers when water in service is free from odor and taste.

Generally the bulk of phyto-plankton in a deep impounding reservoir will be found in the first ten feet of water underlying the surface, but occasionally such growths are uniformly abundant to depths as great as 30 feet. Crustaceans, on the contrary, may not be noticed in the upper water, but found to be swarming at lower levels. Outlet towers are usually provided with gates at vertical intervals of a few feet, permitting draft to be made at any desired point. Plant operation may often be greatly improved by changing the level of draft to secure the best water available. Therefore, samples should be taken at various depths and the organisms enumerated. Other determinations which may be advantageously made on depth and surface samples, are evaluations of amorphous matter resulting from waning plankton pulses, as well as the amounts of

hydrogen sulphide, dissolved oxygen and carbon dioxide. Fluctuations in the amounts of these constituents of raw water materially affect the operation of treatment plants, and definite knowledge regarding them enables the operators to perform their work more intelligently. Other conditions being equal, draft should be made where the oxygen content is at or just below saturation and carbon dioxide at the minimum. One of the most exasperating difficulties in the filtration of water is "air-binding" and this may be due to excessive amounts of carbon dioxide contributed by growths in reservoirs, or, where pre-chlorination is not practiced, by bacterial activity upon non-living organic matter in mixing and sedimentation basins, as well as in the filter units. Stored waters which are allowed to support heavy growths of algae are likely to be supersaturated with free oxygen released in photosynthesis, and this gas is quite as productive of "air binding" as is either entrained atmospheric air or dissolved carbon dioxide. The obvious method of reducing such troubles is by intelligent biological control of the impounding system and treatment plant so as to prevent abnormal accumulations of gases.

REFINEMENTS IN PLANT OPERATION

These are refinements in plant operation which are often neglected, but with increasing demand from the public for water free from odor and taste, the operator who takes pride in the quality of his effluent will find it profitable to avail himself of the information such control can afford.

Chlorination must be regarded as one element of biological control, and to judge by western experiences it is a most important feature of plant operation. The advantages of chlorine as a chemical reagent, either alone or in conjunction with ammonia, in control of phenols for example, though decidedly valuable, may be passed over without further mention, as such reactions are not properly included in a study of biological control. As an economical and ever ready means of suppressing growths within the plant, however pre-chlorination must be accorded first place. Particularly is this true if the raw water happens to be high in organic matter, which tends to foster intense bacterial activity. One western ground water derived from subterranean peats is so high in organic material that bacterial growths within the filter sand are so abundant as to destroy the efficiency of the filters within two or three days unless

continuous prechlorination is practiced, with a distinct residual chlorine content carried to the filter effluent. Many surface waters, and not a few ground water supplies, contain sufficient organic matter to support more or less abundant growths in the filters, and in the majority of instances where caking and short circuiting through the sand are encountered, it will be found that bacterial and fungus growths are the principal causes of the trouble. Prechlorination is a cheap and satisfactory means of control, and its advantages should not be overlooked. In this connection, it is always profitable to make frequent biological studies of sand samples from the different filter units, as well as of wash water, for the information thus derived enables one to avoid impending trouble by appropriate means.

Where activated carbon filters are used as a final stage in treatment, frequent examination of carbon samples, taken preferably at different depths in the bed, are useful, for often bacterial and even mould growths in this unit, rendered possible by removal of chlorine, give rise to "woody" tastes in the final effluent. Such samples are best subjected to microscopic as well as bacteriological examination, especially if iron salts are employed as coagulants. In the latter case an excess of free carbon dioxide, liberated as a result of the coagulation reactions, may result in the effluent retaining a small amount of iron as the ferrous bicarbonate, which, should the spores of Crenothrix find access to the carbon filter, would foster luxuriant growths of this troublesome organism in the carbon, and, worse still, in the clear water well and the distribution system. Those who have experienced the endless troubles caused by such growths do not need to be told the value of biological control: One California city which had unusually serious trouble with Crenothrix in an iron-bearing ground water a few years ago, spent a large sum of money to remove the growths from its distribution system, and has since cheerfully spent each year a small fraction of this sum inscrupulously observed measures designed to prevent a recurrence of the disaster.

EFFECT ON WATER SOFTENING PROCESSES

The water softening process may be influenced by biological control measures, and the effect may be either beneficial or adverse. In a large impounding reservoir storing hard water a heavy growth of phyto plankton which has exhausted the free carbon dioxide will attack bicarbonates of the alkaline earths with a consequent precipitation of calcium and magnesium as the normal carbonates. This is a

natural softening process which may produce noticeable results if the point of draft is close to the surface. Conversely the same vital phenomenon may produce a corresponding increase in hardness if draft is below the thermocline, for the dissolved carbon dioxide present in the lower waters, due to decomposition, will redissolve the precipitated normal carbonates as they settle toward the bottom.

Properly speaking, these phenomena are due to lack of biological control, for where control is exercised no growth should be permitted to reach sufficient magnitude to cause softening, yet such conditions have been observed in California reservoirs.

Another expensive result of failure to control plankton growths has been encountered in an Illinois softening plant which was designed to reuse the lime sludge. Heavy growths of algae which apparently could not be economically controlled, loaded the sludge with odors and tastes to such an extent that its reuse was impossible.

The decay of organisms following a heavy plankton pulse may add sufficient carbon dioxide to the raw water to sharply increase the cost of softening.

The unavoidable conclusion seems to be that biological control is undoubtedly as important a factor in treatment plant operation as is the use of a coagulant, and the more carefully the biology of the various units in the plant and the impounding system is studied, the better and the more uniform will be the quality of water in service. The ability to draw water from any desired stratum should be considered an indispensable element of biological control, and sufficient study of water at all levels should be made to enable intelligent selection of the point from which water for treatment is to be drawn.

DISCUSSION

ARTHUR F. MELLEN:[2] Dr. Wilson's paper contains many valuable suggestions and attacks the problem from a very interesting angle. It will be of particular interest to those who have to do with water whose sources have been influenced by a prolonged period of abnormally low precipitation and gives suggestions which will be helpful in many cases.

We have made routine plankton determinations in our water during the nineteen years the first filtration plant has been in service. This work has taken considerable time and seemed to be of such minor importance that if it had not been for the fact that

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records had been built up over so many years we might have yielded to the temptation to discontinue them. Because of problems of tastes and odors from algae during the past two and one-half years we have been very glad that we have accumulated these records and, therefore, have been able to study the changing flora and their numbers during almost a score of years.

In ordinary years a plankton count of 6000 per cubic centimeter was considered high, although this number gave no particular trouble but we have had during recent years counts in excess of 30,000 per cubic centimeter and while large numbers of those algae forms which are ordinarily considered as being the principal cause of tastes and odors have not been found, we have had considerable trouble of this kind apparently from Cyclotella, Asterionella and Synedra.

Our water is taken from the Mississippi River, but its character, both as to color and plankton, is influenced by the conditions in and the flow from six natural lakes near the headwaters of the river which are controlled by the Government to avoid excessive elevations which at times might cause damage to resorts and other property in the vicinity of these lakes and to provide, on the other hand, the minimum flows required by the barge line, the northern terminus of which is at Minneapolis. In spite of a distance of over 200 miles, these lakes have markedly influenced the character of our water during the recent years of subnormal precipitation. The lake and swamp areas are of such tremendous acreage and the water comes from so many sources that it seems almost hopeless to undertake a campaign of control.

The author states that "practically all organisms entering a treatment plant may be expected to die and disintegrate at an early stage in the treatment." This is not the case in our plants with our usual type of treatment using filter alum as the coagulant and chlorination after filtration. Very high doses of the coagulant will not prevent algae from passing the filters under which conditions tastes and odors are to be expected in the distribution system because of the disintegration of algae and this occurs even in the presence of heavy flows in the mains.

Our problem is principally that of preventing algae from passing the filters. Although we have tried experimentally and on a plant scale every available method of promise for taste and odor removal, the most effective treatment is that of pre-chlorination, usually using from 20 to 25 pounds of chlorine per million gallons. This treat-

ment clumps the algae so that they yield to coagulation and filtration and although it does not give complete control of tastes and odors, it is the most effective method found.

The river water does not approach super-saturation with either free oxygen or carbon dioxide probably because of the loss of these gases during the flow in the river assuming that they are present in large quantities in the reservoir lakes.

The question of the biological condition of the filters may reasonably come under the subject of this paper. The sand in our filters is heavily coated with manganese and contains organic growths. When the problem of severe tastes and odors presented itself the question arose as to whether or not sterile filters would be more effective. Our observation is that the filters in their present condition have a very striking capacity for taste and odor removal. At times, following pre-chlorination, the water applied to the filters has had a very offensive odor which was described as "disagreeable" or "fishy" and a few times a "garlic" odor was noticed. However, the filter effluent under favorable conditions has been reduced to "vegetable" or "grassy" of an intensity of one. Our two plants of 78 and 80 million gallons per day capacity have been in service 19 and 5 years respectively. The older sand is noticeably more effective in taste and odor removal, although it is to be observed that the newer plant has been in service over a period of five years. This has led us to believe that nature furnishes the best control as far as the sand bed is concerned and that it would be unwise to upset the biological conditions now existing.

Unfortunately it is not known whether or not manganese and other inorganic salts present in the sand bed may play a part, directly or indirectly, in taste and odor removal.

ABSTRACTS OF WATER WORKS LITERATURE

FRANK HANNON

The Danger of Indiscriminate Iodine Prophylaxis. J. F. REITH. Pharm. Weekblad, **68**: 77–82, 1931. From Chem. Abst., **25**: 1580, April 10, 1931. Review of arguments against indiscriminate and unsupervised use of iodized salt.—*R. E. Thompson.*

Arsenic Content of the Choussy Well Water, La Bourboule, and Fixation of This Arsenic in the Organism. R. CLOGNE, (Mlle.) A. COURTOIS and CAZALA. Compt. rend., **190**: 1133–4, 1930. From Chem. Abst., **25**: 1608, April 10, 1931. Arsenic was determined by modification of CRIBIER-LÉONARDON method, stain on mercuric chloride paper being fixed by immersion in 10 percent solution of potassium iodide and matched against standard stains prepared under same conditions. Slight excess of potassium permanganate was added to sample to avoid evolution of hydrogen sulfide or phosphine. Arsenic contents of from 5.8 to 6.5 p.p.m. are recorded for season 1928–9, and analyses of tadpoles kept in water at well for from 8 to 10 days showed arsenic contents of 3.25×10^{-3} milligram of arsenic per gram, as against 1.88×10^{-3} when kept in ordinary water. Tadpoles in bottled Choussy water, or in synthetic water of similar composition, contained only 2.73 and 2.325×10^{-3} milligram per gram, respectively.—*R. E. Thompson.*

Determination of the "Chlorinity" of Ocean Waters. THOMAS G. THOMPSON and RICHARD VAN CLEVE. Intern. Fisheries Commission, Univ. Wash. (Seattle), Report No. **3**: 5–14, 1930. From Chem. Abst., **25**: 1608, April 10, 1931. Outline is given of literature dealing with factors that may produce variations between calculated and determined densities. Modifications of MOHR'S method, which differed as to type of volumetric apparatus used, size of sample, and concentration of silver nitrate solution, all gave practically same results. Errors in direct density determinations were found to result from permitting lapse of time between collection and analysis of samples. Determination of "chlorinity" affords best method for calculation of most probably density *in situ* at atmospheric pressure. Table is given for converting "chlorinity" per liter to "chlorinity" per kilogram at 20°.—*R. E. Thompson.*

The Corrosion-Checking Calcium Carbonate Protective Layers in Water Tubes. GERHARD SCHIKORR. Z. angew. Chem., **44**: 40–1, 1931. From Chem. Abst., **25**: 1609, April 10, 1931. Resistance of iron water tubes to rusting is very markedly increased by a calcium carbonate protective layer. For formation of layer, definite quantity of dissolved oxygen must be present. Water

must contain sufficient dissolved calcium carbonate and must not contain too much free carbon dioxide. Action of oxygen may be traced to electrochemical effect, in contrast to theory of TILLMANS, HIRSCH, and SCHILLING (C. A., **23**: 1973). In order to demonstrate that formation of protective layer is not the result of adsorption of carbonic acid by ferric hydroxide, but is a cathode reaction, a noble metal was used in place of iron. Experiments indicate that with suitable arrangements calcium carbonate can be precipitated from calcium bicarbonate solution on platinum with aid of oxygen.—*R. E. Thompson.*

The Continuous Control of the Salt Content of Boiler-Feed Water by Means of a Visual Conductivity Measurement. E. ROTHER and G. JANDER. Z. angew. Chem., **43**: 952–4, 1930. From Chem. Abst., **25**: 1609, April 10, 1931. Chloride content of feed water is determined by leading water through specially designed tube containing platinum electrodes. Determinations are continuously recorded potentiometrically.—*R. E. Thompson.*

Determination of pH in the Estimation of the Productivity of Carp Ponds. LOUIS ROULE. Compt.-rend. acad. agr. France, **16**: 1056–60, 1930. From Chem. Abst., **25**: 1610, April 10, 1931. Reaction of water should be between pH 6 and 8 and care must be taken to maintain it within these limits. Tests and adjustments of reaction should always be made early in morning and in shade, as sunlight has very marked effect on chemical changes taking place.—*R. E. Thompson.*

Liège Water Service, 1830–1930. E. THIRIAR. Tech. sanit. munic., **25**: 370–5, 1930. From Chem. Abst., **25**: 1606, April 10, 1931. An historical account.—*R. E. Thompson.*

Fluorescence of Water in Filtered Ultra-Violet Light as an Indicator of Pollution. H. IVEKOVIC. Z. Hyg. Infektionskrankh., **112**: 54–61, 1931. From Chem. Abst., **25**: 1607, April 10, 1931. Fluorescence of drinking waters was examined in light from quartz lamp passing through a black glass filter of nickel oxide, and compared with gelatin standards. Many waters showed bluish white fluorescence, intensity being proportional to consumption of permanganate. Organic matter present is considered to be cause of fluorescence.—*R. E. Thompson.*

A Limnological Significance of Free Carbonic Acid. E. LINDEMANN. Naturwissenschaften, **18**: 1113, 1930. From Chem. Abst., **25**: 1606, April 10, 1931. Evidence is advanced regarding importance of carbon dioxide content of water for growth of *Peridinia*, provided pH is favorable.—*R. E. Thompson.*

Investigation of Calcium Hypochlorite (65 percent Available Chlorine). BERNARD MELKON. Am. J. Pharm., **103**: 46, 1931. From Chem. Abst., **25**: 1634, April 10, 1931. Analysis of delivery of product marketed under trade title H. T. H. by Mathiesen Alkali Works, Inc., showed: insoluble in water, 8.40; silica, 0.28; ferric oxide, 0.38; aluminum oxide, 1.76; calcium oxide, 32.17; available chlorine 62.70 percent.—*R. E. Thompson.*

Commercial Paints: Substitutes for White Lead and Anti-Corrosion Paints. A. VILA. Chimie & industrie, **24**: 1052–67, 1930; cf. C. A., **22**: 3998. From Chem. Abst., **25**: 1688, April, 10, 1931. Various accelerated aging tests for anti-rust paints were tried, but rapidity of paint deterioration was in no case more, and in some cases less, than under exposure to atmosphere, or to fresh, or sea water. As value of these paints depends essentially on their impermeability, following test is suggested. Apply two coats of sample over whole of small aluminum plate and allow to dry normally. Fill hollow of plate with solution of sodium chloride and mercuric chloride in distilled water and let stand at atmospheric temperature; if coating is impermeable, solution evaporates without producing corrosion, but if not, mercuric chloride reacts vigorously with aluminum as soon as solution has penetrated to metal. Tests showed that a number of French anticorrosion paints containing no lead salts are fully equal to, or better than, the old minium paints.—*R. E. Thompson.*

Rapid Method for Determination of Benzene and Phenol in Ammoniacal and Waste Liquors. W. MÜNZ. Brennstoff-Chem., **12**: 3–4, 1931. From Chem. Abst., **25**: 1660, April 10, 1931. To determine phenol, measure into 250-cc. distillation flask 25 cc. of filtered liquor. Add ammoniacal copper sulfate solution (prepared by adding concentrated ammonium hydroxide in excess to cold saturated copper sulfate solution) in excess of precipitation and then 5 cc. 40 per cent sodium hydroxide solution. Boil over free flame, with shaking, until ammonia vapors are no longer evident, connect flask to condenser, and pass carbon dioxide through solution at rate of 2 or 3 bubbles per second. Neutralization should be complete in about 5 minutes. Distil to dryness in slow stream of carbon dioxide. Rinse condenser, adapter, etc. into 250-cc. volumetric flask used as receiver and make up distillate to volume. For titration, 25, or 50, cc. of this is used in case of ammonia liquors, but with waste liquors the whole may be necessary. Add, in stoppered Erlenmeyer flask, 25 cc. 0.1 normal potassium bromide-potassium bromate solution and then 15 cc. 15 percent sulfuric acid. After one-half hour add 10 cc. 10 percent potassium iodide solution, let stand one-quarter hour, and titrate excess, using starch indicator, with 0.1 normal sodium thiosulfate solution.—*R. E. Thompson.*

The Nitrogenous Compounds in Sea Water. Methods for Their Study. J. GIRAL. Rev. acad. ciencias Madrid (2), **25**: 10, 203–19, 1930. From Chem. Abst., **25**: 1926, April 20, 1931. Attention is called to wide variety of types of nitrogenous compounds in sea water. Review is given of existing methods which might be applied to study of following classes of nitrogen compounds: free nitrogen, ammonia, nitrites, nitrates, albumins, total nitrogen, and organic nitrogen compounds other than those mentioned. No experimental data are given and no new methods are suggested.—*R. E. Thompson.*

Correlation of Sieving Analyses. H. HEYWOOD. J. Inst. Fuel, **3**: 428–32, 1930. From Chem., Abst., **25**: 1714, April 20, 1931. Variable factors entering into sieve analysis are: personal factor, nature of material, time, weight of material, variation in sieve aperture. These factors are discussed.—*R. E. Thompson.*

Electrolytic Cells for Chlorine and Caustic Soda. C. L. MANTELL. Chem. Met. Eng., **38**: 88–90, 1931. From Chem. Abst., **25**: 1740, April 20, 1931. Construction details of ten makes of electrolytic cells are briefly described and compared, and tabulated data, compiled from information furnished by users of various cells, are given. Yields of from 0.62 to 0.76 pound chlorine per kilowatt-hour are common practice.—*R. E. Thompson.*

Direct Green B. A New Reagent Very Sensitive to Copper. P. SISLEY and DAVID. Bull. soc. chim. (4), **47**: 1188–92, 1930. From Chem. Abst., **25**: 1756, April 20, 1931. When copper salts are added to solution of direct green B at 90°, color changes from blue-green to violet-pink. Solution should have pH value between 6 and 9 and dye: copper ratio should be 10; but satisfactory results are obtained when ratio is between 6 and 50. Reaction is sensitive to 0.1 milligram of copper per liter. Anions have no effect and no other metal was found that gave violet-pink color. Neutral salts retard the reaction and reduce color intensity, potassium sulfate having greatest effect. Colloids mask the reaction.—*R. E. Thompson.*

Subterranean Water Conditions in the Coastal Regions of the Netherlands. J. VERSLUYS. Econ. Geol., **26**: 65–95, 1931. From Chem. Abst., **25**: 1773, April 20, 1931. Geologic history of area is given and character of deposits and their influence on ground water are described. Chemical action in subsoil of dunes is discussed. Formation of water with excess of alkalies is explained by assuming that sea water has ceded alkalies to the soil, and that these, when sea water was displaced by calcareous fresh water, have in turn been exchanged for calcium.—*R. E. Thompson.*

New Volumetric Method for the Estimation of Sodium. A. BLENKINSOP. J. Agr. Sci., **20**: 511–6, 1930. From Chem. Abst., **25**: 1759, Arpil 20, 1931. Certain disadvantages of gravimetric sodium zinc uranyl acetate method (C. A., **22**: 2898; **23**: 462; **24**: 5912), due to manipulative details, are overcome by proposed method in which uranium is determined by reduction to uranous state by means of standard titanium trichloride solution. Precipitate sodium as $(UO_2)_3ZnNaAc_9 \cdot 6H_2O$, as in gravimetric method, filter through a Jena-glass filtering funnel (porosity G4), remove as much of reagent as possible by suction, and wash 2 or 3 times with 2-cc. portions of saturated solution of precipitate in 95 percent alcohol. Dissolve precipitate in dilute hydrochloric acid, transfer to 500-cc. conical flask, add about 10 cc. of strong hydrochloric acid and displace air in flask by stream of carbon dioxide. Reduce uranium to uranous state by addition of excess of 0.02 normal titanium trichloride solution. Maintaining carbon dioxide atmosphere, shake mixture and allow to stand 2 or 3 minutes, or heat to 40–50° to insure complete reduction. Add from 20 to 25 cc. of 2 per cent hydrofluoric acid to stabilize, or even precipitate, the uranium. After addition of about 10 cc. of 10 per cent potassium thiocyanate, estimate excess titanium trichloride by back titration with 0.02 normal ferric alum to permanent red color.—*R. E. Thompson.*

Supplementary Discussion of the Drop Experiments of Evans. E. MAAS and E. LIEBREICH. Korrosion Metallschutz, **6**: 172–3, 1930. From Chem.

Abst., 25: 1779, April 20, 1931. Reiteration of belief in convection currents as explanation of phenomena observed by EVANS (cf. C. A., 24: 3744, 5705).—*R. E. Thompson.*

The Vibriocidal Powers of the Water of Certain Rivers of India. SARANJAM KHAN. Indian J. Med. Research, 18: 361–5, 1930. From Chem. Abst., 25: 1864, April 20, 1931. Water of Ganges and Jumna rivers has vibriocidal properties which are completely destroyed by heating to 55° for one-half hour.—*R. E. Thompson.*

Dependence of Length of Life of Household Pipe Connections upon Material, Size of Bore, and Water Quality. A. F. MEYER. Gas- und Wasserfach, 74: 30, 703–710, July 25, 1931. Paper read at meeting of Deutscher Verein von Gas- und Wasserfachmännern, June, 1931. Chemnitz water supply contains on average 27.3 p.p.m. of chloride, 26.2 p.p.m. of free carbon dioxide, 71 p.p.m. of total hardness, and pH of 6.8. Formerly treated with ozone, since 1925 it has been chlorinated. Water has strong action on lead and zinc, so that only iron, or copper, pipes are used. Prior to 1880, service pipes were of 1-inch bore cast iron; this was increased to 1½-inch and, since 1913, wrought iron 2-inch pipe is used and is preferable. Chemnitz ground water is more aggressive than impounded water and contains more solids, hardness, iron, and manganese.—*W. G. Carey.*

Bacteriological and Chemical Water Treatment at Breslau. W. WAGENKNECHT. Gas- und Wasserfach, 74: 29, 673–678, July 18, 1931. Presented at meeting of Deutscher Verein von Gas- und Wasserfachmännern, June, 1931. Supply is ground water, some natural, some bank filtered, and some earth filtered into seepage basins 160 to 200 feet from water source. If necessary, supply can be augmented from river Oder. Ordinarily water contains very few bacteria; but severe frost of 1928–29 interfered with ground filtration and high bacterial and coli figures resulted. Water is of medium hardness and contains sulphuretted hydrogen, ammonia, iron in organic and inorganic forms, manganese, and free carbon dioxide. It is treated by aëration, lime water, sedimentation, and rapid and slow filtration. River Oder water is very polluted and, in summer, highly colored; it is treated by prechlorination, addition of alum and lime, and sedimentation. As it was found that filtration of water containing free chlorine led to increased bacterial content, water is now dechlorinated prior to rapid and slow filtration.—*W. G. Carey.*

Purification of Water. Chemistry and Industry, 50: B. 656, July 10, 1931. Apparatus is described in U. S. Patent 1,782,850 of 25.11.30, in which tastes, odors, etc., may be removed from water by gently agitating with from 0.05 to 0.2 percent of powdered activated carbon and then allowing settlement in a vessel in which separation of finest carbon particles is facilitated by maintaining a rotational movement of 1 or 2 turns per hour. Carbon sludge removed may be re-used, after revivification if necessary. Plant is suitable for batch, or continuous, treatment.—*W. G. Carey.*

Application of Piperidinium Piperidyl-Dithioformate to the Colorimetric Determination of Copper. R. HARRY. Analyst, **56**: 688, 736–737, November, 1931. By yellowish brown color produced with alcoholic solution of piperidinium piperidyl-dithioformate, as little as 0.00001 gram copper may be determined: color is stable and unaffected by slight excess of acid or alkali. Metals of third group are removed by precipitating the copper from acid solution by hydrogen sulphide, filtering, washing with water containing hydrogen sulphide and acetic acid, heating with fuming nitric acid, washing with water, evaporating and dissolving in water. Organic matter is oxidised by ashing. Suitable portions are treated with 1 cc. of 0.1 percent solution of the reagent and compared with standard.—*W. G. Carey.*

Volumetric Determination of Sulphate in Water. B. PASCHKE. Zeitschrift für Untersuchung der Lebensmittel, **62**: 378, 1931. Volume containing from 0.02 to 0.04 gram of sulphate radical is acidified with 0.1 normal hydrochloric acid, evaporated, treated with excess 0.2 normal barium chloride, boiled, and cooled to 26°C. Sodium chloride, ethyl alcohol, and sodium rhodizonate are added and mixture titrated with 0.2 normal sulphuric acid until all barium rhodizonate is converted to barium sulphate, when the flocculent red suspension changes through yellow to colorless. Then 8 times cc. of barium chloride used equals sulphur trioxide in milligrams.—*W. G. Carey.*

De-acidification of Drinking Water. R. SCHMIDT. Gas- and Wasserfach, **74**: 27, 635–637, July 4, 1931. Water with calcium carbonate content of 107 parts per million, or more, can be de-acidified by aëration; if iron is also to be removed, sufficient oxygen must be present to prevent solution of iron in pipes; and in closed plants using compressors additional aëration and release of pressure by spraying into reservoir are recommended. Filtration through marble is effective, but a protective layer is formed in pipes only if calcium carbonate after such treatment is 72 parts per million, or more. Soft waters must have high free carbon dioxide content for this; so that aëration to remove carbon dioxide from such water is inexpedient; with low free carbon dioxide in very soft waters aggressive action is only decreased and preliminary removal of suspended matter and iron is necessary. Iron in soft waters cannot be removed by aëration and filtration, so that marble filters to remove iron should precede de-acidification filter. Formation of protective layer is only effective with marble within narrow limits; but for reduction of aggressive action, range is greater. Where aëration, or marble filtration, fails, de-acidification with lime water is usually successful; but careful dosing and control are needed.—*W. G. Carey.*

High Salt Content of City Water (Dessau, Germany). B. SZELINSKI. Gas- und Wasserfach, **74**: 34, 801, August 22, 1931. Water contains 540 parts per million of chlorine as chloride, 377 of sulphate (SO_3), 320 of lime, and 74.2 of magnesia. High chloride is not uncommon in regions with saline subsoil and does not necessarily indicate sewage. Freezing point determination is important to detect alteration in total content of salt ions.—*W. G. Carey.*

Information Concerning the Removal of Odor and Taste from Artificial Ground Water. VOLLMAR. Gas- und Wasserfach, **74:** 35, 805–810, August 29, 1931. Presented at meeting of Deutscher Verein von Gas- und Wasserfachmännern, June 1931. Dresden supply is augmented by water pumped from Elbe, which is settled for four hours, passed through rapid filters to seepage basins, and, after filtration through about 250 feet of natural soil, received in wells. Agar count is thereby reduced from 1340 to 3 per cubic centimeter, and *B. coli* are eliminated; but tastes and odors develop owing to growth of algae on floors of seepage basins. Treatment with chlorine was unsuccessful, as was permanganate; but either filtration through vessel containing Hydraffin active carbon, or aëration by spraying, was efficacious At another Dresden works, taste and odor due to removal of oxygen from Elbe water by bank filtration was remedied by aëration.—*W. G. Carey.*

Removal of Iron, Manganese, and Phenol from Water by the A. D. M. Process. OEFVERBERG. Gas- und Wasserfach, **74:** 34, 787–792, August 22, 1931. Presented at meeting of Deutscher Verein von Gas- und Wasserfachmännern, June 1931. Owing to increased water demand in Ludgwigshafen, wells with high bacterial content and containing phenol, iron, and manganese were purified by the ADLER Diachlor-Mutonite process. Water is treated with 3 parts per million of chlorine, iron being precipitated as ferric hydroxide, manganese as manganese dioxide, and any sulphuretted hydrogen as free sulphur. All such impurities are retained by the "diachlor" filter, from which water passes to contact filter filled with specially prepared manganese dioxide, which removes humins, last traces of manganese, and bacteria. Excess chlorine is removed by "Mutonator" which is vessel filled with activated carbon of about 4 millimeters diameter. "Diachlor" filters are cleaned every month and yield a dark brown slime of manganese dioxide and ferric hydroxide. Phenol is also eliminated in the process, no trace being detected in the effluent from the "Mutonator."—*W. G. Carey.*

Experiments on the Importance of Protozoa in the Self-Purification of Water. K. KYRIASIDES. Gas- und Wasserfach, **74:** 38, 891, September 19, 1931. Typhoid and cholera germs added to unsterilized tap water and to river Spree water were not detectable after from 4 to 6 days, nor in well water, after from 13 to 16 days. In same waters sterilized, disappearance took longer, possibly due to absence of protozoa. When pure protozoa cultures were added to sterilized waters containing bacteria, both types were absent after 48 hours in both tap and river waters; in well water, cholera vibrio was undetected after 7 days and *B. typhosus* after 14 days. Protozoa seem therefore to hasten destruction of pathogenic bacteria.—*W. G. Carey.*

Border Cities Water Supply. Canadian Engineer, **60:** 4, 18, January 27, 1930. It is estimated that the proposed $554,000 extension to the Essex Border Utilities Commission filter plant will supply the needs of the various municipalities for the next 10 years. Windsor is appealing her apportionment of the capital cost of the present and proposed plants, it being maintained that

Windsor's water consumption is steadily decreasing and that any increased water consumption is due to developments in the other municipalities. Windsor is also opposing the construction of the new filter plant on the grounds that it is not necessary.—*A. W. Blohm* (*Courtesy U. S. P. H. Eng. Abstracts*).

Water Works and Sewerage in Ontario. A. E. BERRY. Canadian Engineer, 59: 10, 291, September 2, 1930. A review of the activities of the Sanitary Engineering Division in the fields of recreational sanitation (tourist camps and refreshment booths) milk control, water supply and sewerage, with particular reference to latter two. Over 80 percent of the water distributed from the various systems for domestic purposes is chlorinated and 35 percent is filtered. Approximately 350,000,000 gallons of water per day are delivered by these systems. There are in operation 152 chlorination plants. Ortho-tolidin is supplied free of charge to municipalities employing chlorination and the operators are urged to make frequent tests and to keep adequate records. The desirability of providing duplicate chlorination equipment is stressed.—*A. W. Blohm* (*Courtesy U. S. P. H. Eng. Abstracts*).

The North Holland Provincial Waterworks. J. VAN OLDENBORGH and others. Water en Gas, 163–17, 173–194, September 5–19, 1930. Water and Water Engineering, 32: 383, 544, November 20, 1930. This undertaking supplies 121 communes on the mainland and five on the islands. There are 1,582 km. of mains and 79,187 service connections, and average daily consumption of 26,000 cu.m. 58 communes are supplied by the waterworks at Bergen. The gathering ground is dune land, with fine and coarse sand and shell débris. A range of 60 wells extends over a mile, the older wells shallow, but the newer additions 140 to 150 m. deep, with 4 in. split copper filter pipes 14 m. long, and suction pipes provided with foot valves. The pumps are electric driven, of total 1,200 cu.m. per hour capacity, to a head of 21.5 m., and three high-pressure pumps of 1,400 cu.m. total capacity, to heads of 61 to 70 m. They deliver to a 500 mm. Bonna main. A total filter surface of 5,000 sq. m. is available. Over each division of filters are two sprayers of the bronze ring type. There were three water towers of 1,400 cu. m. total capacity, and low reservoirs of 2,000 cu. m. at Hoorn. A new tower at Wieringerwaard has 1,000 cu. m. capacity, at a height of 27.88 m., and is constructed with 12 inner and 12 outer columns of reinforced concrete, with concentric tanks of 400 and 600 cu. m. capacities.—*A. W. Blohm* (*Courtesy U. S. P. H. Eng. Abstracts*).

Water Supply in the Madras Presidency. Surveyor, 79: 2042, 332, March 13, 1931. The Madras Presidency has a population of over 42 million, of which approximately 3 million is classed as urban. About 69 percent of the urban and 0.1 percent of the rural population now is or shortly will be supplied with water from protected sources. While progress is being made slowly the work is retarded by lack of funds due to: limitations in local taxation and consequently limited borrowing power; inadequate funds from provincial revenue; and the scarcity of professional engineers to advise local authorities on proposed schemes.—*A. W. Blohm* (*Courtesy U. S. P. H. Eng. Abstracts*).

NEW BOOK

Journal of the Southeastern Section, A. W. W. A. 2: 96 pp., 1932. Advances and Developments in Powdered Activated Carbon Application in Water Treatment. F. E. STUART. 5–16. Review of various applications of powdered activated carbon in several water purification plants during last few years. Powdered activated carbon came into use to remove from water objectionable tastes and odors due to phenolic substances and micro-organisms. Observations indicate that sedimentation basin and softening plant sludge can be deodorized with powdered activated carbon. When powdered carbon is fed with aluminum sulfate, coagulation is improved and in some cases amount of coagulant is reduced due to improved floc. Brief description of physical and chemical nature of activated carbon and of its adsorptive action on tastes and odors is given. In author's discussion, claim is made that residual alumina is reduced when carbon is used at time of coagulation. Application of carbon ahead of prechlorination reduces amount of chlorine necessary to maintain a given residual. Adsorption of organic matter by the carbon particles acts as balance wheel in controlling fluctuations in chlorine demand. Powdered activated carbon is not effective in removing chloramine residual. **Turbidimeters in Use at the Atlanta Filtration Plants.** PAUL L. WEIR. 17–23. General description of various pieces of conventional laboratory equipment for turbidity measurement and floc detection in use in Atlanta plants. Detailed description of equipment developed at Atlanta plants for accurately measuring turbidities between 1 and 25 p.p.m., using standard suspension bottles, is given, including dimensioned drawing. Tables are given for preparing by dilution standard turbidity suspensions, with raw river water between 5 and 25 p.p.m. and with fullers' earth between 1 and 5 p.p.m. **Wells and the Part they Play in our Water Supply.** W. R. MCGREW. 24–34. Discussion of well drilling from driller's point of view. Much of difficulty encountered with unsatisfactorily drilled wells can be avoided by contracting with responsible drillers. Suggestions for keeping well logs and for licensing of drillers by State are made. Drainage wells should be regulated by state to reduce chances of contamination of underground supplies. Uncapped abandoned flowing wells rapidly deplete underground supply and reduce static pressure. State regulation of this careless practice would prove economical. **Water Softening Plant, Thomasville, Ga.** LOWELL CADY. 35–44. Lime-soda process is used in softening water from deep wells at Thomasville, Ga. Plant consists of two mechanical agitators, one clarifier, three carbonating chambers, one settling basin, and two gravity filters. Over-treatment with lime to caustic alkalinity of 40 p.p.m. and re-carbonation is practised. Ground quicklime passing ½-inch mesh is fed into continuous slaker. Resulting lime suspension, together with soda ash solution, is applied to raw water in agitators. After settling in Dorr clarifier, aluminum sulfate is added and water passes to chamber into which carbon dioxide is blown, which effects both mixing and carbonation. The precipitate, assisted by alumina, settles in sedimentation basin and water passes through second carbonating chamber to filters. Filter effluent is led through third carbonating chamber, where carbon dioxide produces stability, before being stored in finished water reservoir. Carbon dioxide is

obtained from nearby municipal power house stack and passed through wet and dry scrubbers. Total hardness is reduced from 12 to 4 grains per gallon at cost, including operating and fixed charges, of $39.05 per million gallons. **Our Streams and their Flow.** E. D. BURCHARD. 45–60. Discussion of volumetric characteristics of stream flow as shown by records of U. S. Geological Survey. Wide differences in maximum, mean, and minimum unit run-off for typical streams in Southeast illustrate danger of estimating these values from formula and shows necessity for adequate and continuous stream flow records. **The Water Supply of Spartanburg, S. C. and Its By-Products.** R. B. SIMMS and J. K. MARQUIS. 61–68. The by-products of this municipally-owned utility, which foster continued good will of public, are (1) generation and sale of hydro-electric power; (2) sale of water to industries during off-peak loads; and (3) promotion of recreation and health through operation of swimming pool and park on water plant property. **Water Works Accounting.** HILARY H. GARDNER. 69–73. Discussion of accounting as tool at service of water works manager, or superintendent. **Information on Water and Sewage Works Problems: Where It can be Found.** LINN H. ENSLOW. 74–84. Description of sources and methods available for securing information of value to water and sewage works superintendents and operators. Excellent lists of reference books, pamphlets, bulletins, and reports, showing authors and publishers, covering fields of water supply, sewerage, and sewage disposal, are included. Simple scheme for filing information taken from current Journals is suggested. **Some Trends of Municipal Sewage Disposal Practice.** WELLINGTON DONALDSON. 85–96. There is no standardized process, or combination of processes, suitable for all sewage disposal conditions. As functions of various treatment steps are becoming better understood, there is tendency to separate processes and thereby secure better results. This segregation of functions leads naturally to use of much mechanical equipment. Review of present practice regarding utilization of various treatment units and equipment is given. Brief description of the Laughlin Process involving use of lime, paper pulp, and coagulant is included. In **discussion** of this paper, L. H. ENSLOW outlines lime-chlorine treatment of sewage.—*W. H. Weir.*